Fire Resistance of Epoxy-grouted Steel Rod Connections in Laminated Veneer Lumber (LVL)

by

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Fire Engineering Research Report 04/7

June 2004

A thesis submitted in partial fulfilment of the requirements for the degree of Master of Engineering in Fire Engineering

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Abstract

Epoxy-grouted steel rods are becoming increasingly popular for connections in structural timber in glue-laminated timber (glulam) and laminated veneer lumber (LVL). This research focuses on the fire resistance of these connections in LVL timber. These connections have been found to have high strength under service temperatures, but epoxy is known to soften at relatively low temperatures.

To determine connection performance, an experimental investigation was carried out on the axial tensile strength of connections that utilised a threaded steel rod bonded into the timber using two epoxy resins and a composite adhesive. Some specimens were tested at constant elevated temperatures while similar specimens were tested in simulated fire conditions under constant load. The three adhesives tested gave different connection strengths at ambient temperatures and showed different strength losses at elevated temperatures.

Acknowledgements

Financial support for this project was provided by Carter Holt Harvey Futurebuild and the Foundation for Research, Science and Technology in the form of an Enterprise Scholarship and the New Zealand Timber Design Society. Thank you to Hank Bier and Ross Davison of Carter Holt Harvey for their technical support throughout the project.

Thank you to my supervisors Andy Buchanan and Peter Moss for their support and dedication to this project. This project could not have run so smoothly without the expertise and support of the Civil Engineering technicians. Special thanks to Grant Dunlop, Russel Peoples, Kevin Wines and Richard Newton.

Adhesives for this project were kindly donated by Hilti Adhesives and Adhesives Technology. Thank you to Darko Arandjelovic (Hilti) and Chris Timms (Adhesives Technology) who sadly passed away prior to the completion of this thesis.

A large thank you to my wife, Adie Fortune, for keeping me motivated throughout and for always supporting me through this research. To Clare-Ann Fortune your patient proof reading and red pen was enormously appreciated. To my family, the Fortune family and friends for the light relief when the meltdowns occurred and encouraging me to keep going.

The photos in section 3 are from the collection of Andy Buchanan, unless noted otherwise. The original photographers of these photos are unknown.

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1 Introduction

Timber structures in New Zealand are commonly used for low rise residential dwellings. Another use for which they are suitable is single storey buildings with large clear spans (e.g. swimming pools, gymnasia and halls). Timber's light weight and good strength make it suitable for this type of application.

To achieve large spans, deep sections are needed. Modern forestry does not provide sawn timber in the dimensions which are required for these spans and so other methods such as glue laminated timber (glulam), or laminated veneer lumber (LVL) are typically used to manufacture structural members with large cross-sections.

These timber members carry significant loads and hence connections within these members need to be very strong. Rigidity is also important for moment resisting frames, as small rotations at the connections of a large building may give considerable deflections. Increasingly connections in glulam and, to a lesser extent, LVL are making use of steel rods epoxy-grouted into the timber. Connections using epoxy-grouted steel rods provide the strength and rigidity required, but there is uncertainty as to the fire resistance of these connections.

Heavy timber construction has good fire performance due to its ability to char. This provides insulation to the inner portion of the member, where strength is retained. Due to this insulation, heavy timber construction often performs better in fire than unprotected steel and some concrete structures.

Epoxies are well-suited to connections using glued in rods as they provide a strong, rigid bond. They are also easy to assemble and as the connection is typically hidden it has aesthetic appeal. Epoxy has a major potential drawback in structural connections in that it loses strength at relatively low temperatures.

The effect of heat upon the epoxy-grouted steel rod connection will be investigated in this research using LVL timber. The objective of this is to determine the fire resistance of epoxy-grouted steel rod connections in LVL timber. Considerable research has been undertaken at the University of Canterbury investigating the performance of epoxy-grouted steel rod connections under cold conditions. This research is intended to build on that work.

2 Objectives

2.1 Overall aims

The primary objective of this research was to quantify and improve the fire resistance of the epoxy-grouted steel rod connection in LVL timber.

Secondary objectives of this research were:

- To investigate alternative adhesives, with greater fire resistance to use in place of epoxy resin for grouting steel rods into LVL

- To investigate whether the addition of a mechanical bond would increase the fire resistance of epoxy-grouted steel rod connections

- To determine the performance of epoxy-grouted steel rod connections in LVL at constant temperatures

- To determine the performance at ambient temperatures of epoxy-grouted steel rod connections in LVL which had previously been exposed to high temperatures

- To determine the behaviour of epoxy-grouted steel rod connections in LVL when exposed to simulated fire conditions

2.2 Background

LVL is a relatively new high strength structural timber product in New Zealand. To enable the high strength of LVL to be utilised high strength connections are required. Epoxy-grouted steel rod connections have been shown to have high strength in LVL (van Houtte, 2003) and glulam (Deng, 1997 and others) and have been used successfully in many glulam structures (McIntosh, 1989, Buchanan and Fletcher, 1989). Only limited research has been undertaken in the evaluation of fire resistance of epoxy-grouted steel rod connections (Barber, 1994).

Carter Holt Harvey Futurebuild has supported this research, both directly and indirectly through a Bright Futures Enterprise scholarship. Carter Holt are a major producer of LVL in New Zealand and the development of a fire resistant connection for LVL would enable them to increase the applications for which LVL can be used. This would result in greater sales for Carter Holt.

Intended outcomes of this research were to determine the fire resistance of the epoxy-grouted steel rod connection in LVL timber and to assess whether alternative adhesives are available which could be used in place of the epoxy resin to improve the fire resistance.

2.3 Methods

There are three distinct phases within the experimental part of this research. Each phase has its own objectives. Each of these three phases concentrates on one component of the development and calculation of the fire resistance of connections using epoxy bars in LVL timber.

2.3.1 Phase one – cold testing

Previous research performed on grouting steel bars inserted into timber has typically used epoxy resin or polyurethane adhesives. In this research West System, an epoxy resin proven for this type of connection with glulam timber (Gaunt, 1998) and also in LVL timber (van Houtte, 2003) was used along with two alternative adhesives. These alternative adhesives are an epoxy resin (RE 500, by Hilti) and a hybrid urethane methacrylate/cement adhesive (HY 150, by Hilti).

RE 500 was chosen as it was predicted that it had better strength retention at elevated temperatures than the West System. HY 150 was chosen as it is used extensively in industry for grouting steel bars into concrete where fire resistance is required. The strength of HY 150 as an adhesive in timber was uncertain.

In phase one of testing the performance of the two Hilti adhesives were compared against the benchmark West System to evaluate their suitability in this connection system. Comparisons were made between both the ultimate loads and the failure modes in order to compare the behaviour and strength of the three alternative adhesives.

2.3.2 Phase two –oven testing

It is a well accepted weakness of epoxy resins, especially those cured at room temperature, that they begin to lose their strength at relatively low temperatures (Shields, 1984). This phase of testing aimed to provide an evaluation of the strength of the connections at a range of temperatures using each of the three adhesives.

A second component of this phase of testing investigated the potential post-fire strength of the connections. Heavy timber construction can often be re-used after a minor fire, but if the connections are severely damaged then this would require the structure to be replaced. This research investigates the long-term effects of fire on the adhesives effectiveness by heating and cooling specimens prior to tensile testing. By comparing the results from these cooling tests with those established in tests at ambient temperatures any strength loss as a result of the heating process could be identified.

2.3.3 Phase three – furnace testing

This phase was an application of phases one and two of the research. In these tests the connections were held under a constant tensile load while they were exposed to an external heat flux in a furnace. The objective of these tests was to evaluate the fire performance of the connection. Quantification of the fire resistance was made by comparing the results in the test furnace to those achieved burning LVL in a furnace exposed to the ISO 834 fire by Lane (2004).

3 Literature review

Public buildings such as swimming pools, halls and gymnasia often call for large clear spans. Modern timber manufacturing techniques such as creating glued laminated timber (glulam) and LVL allow for these spans to be achieved, but the high bending moments generated in moment-resisting frames using these sections make the design of connections difficult.

The design of these connections becomes further complicated when there is a need for the connection to have a fire resistance.

3.1 Current technology

There are various technologies currently available in New Zealand for the moment-resisting connections in large LVL sections in portal frames. Some of the more common are described in Buchanan (2002) and include nailed gusset plates, steel gusset plates and epoxied rods with a steel hub. Some of these are shown in Figure 3.1 and are described in sections 3.1.1 through 3.1.5.

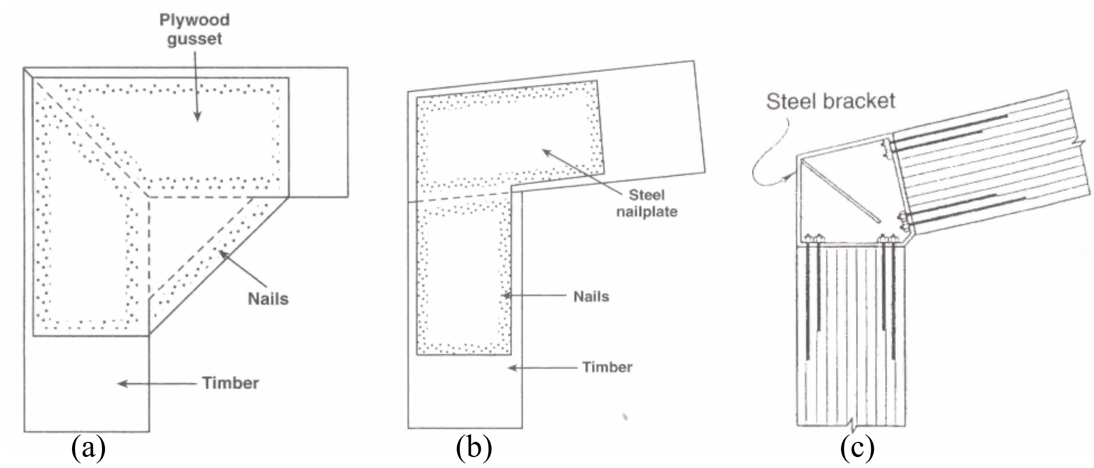


Figure 3.1 Types of portal connections: (a) plywood gusset, (b) steel nailplate, and (c) epoxy rods with steel hub. Reproduced from Buchanan (2002)

3.1.1 Nailed connections

Nailed connections in heavy timber members typically involve sheets of plywood or steel either side of the intersection of two members. These connections are simple and are commonly used in portal frame buildings in New Zealand. These plywood sheets are fixed in place with rows of nails to form a rigid connection, as shown in Figure 3.2

Figure 3.2 Plywood gusset plate connection

Nailed connections using plywood gussets are simple to construct as they do not require any lining up of holes, or predrilling. They can also be constructed on site using a pneumatic nail gun. The strength of the gusset plate is limited by the thickness of the plywood and also by the plywood sheet size (1200 x 2400 mm). Typically this limits the span of the portal frame to around 20m (Buchanan and Fairweather, 1993). Design guidance for nailed gusset plates is given by Buchanan (2002).

Steel gusset plates can also be used. These have the advantage over plywood gusset plates of unrestricted size and thickness. Steel gusset plates are used in a similar manner to plywood and are attached with rows of nails, as shown in Figure 3.3. As steel is too hard to nail through, the holes in a steel gusset plate must be pre-drilled before assembly. Design of steel gusset plates is similar to those for plywood gusset plates and is also given by Buchanan (2002).

Figure 3.3 Steel gusset plate

Connections can also be made using toothed metal nailplates, although these have fairly low capacity and are mostly limited to domestic trusses. These are simple to use as the plates are lined up with the timber members and then pressed on using a hammer.

3.1.2 Bolted connections

Bolts can provide very strong connections, using fewer fasteners than nailed connections. Bolted connections range from simple types with bolts passing through holes in each member (as shown in Figure 3.4), or bolted connections may use external steel plates to transmit forces between members, or steel rings placed between the members (as shown in Figure 3.5).

Figure 3.4 Bolted connection

Bolted connections typically use slightly oversized holes for ease of construction, but this reduces their stiffness. The toothed connector plates shown in Figure 3.5 are sometimes used to increase the strength of the connection by increasing the friction between the two surfaces (Larsen, 2003).

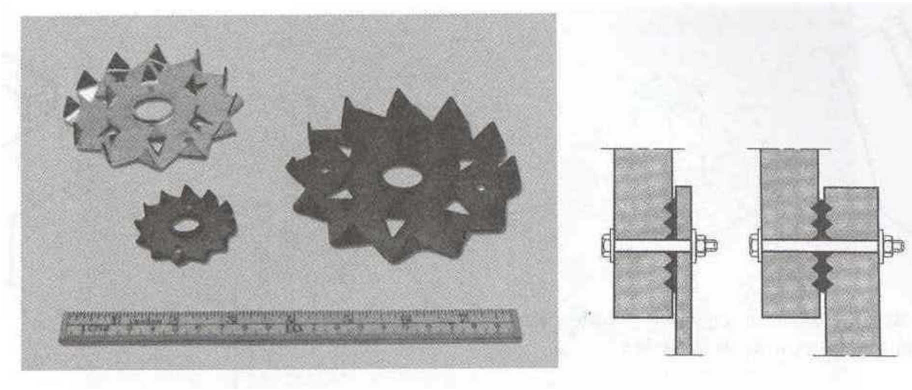


Figure 3.5 Toothed ring connectors (Larsen, 2003)

3.1.3 Glued connections

Glued connections are popular in glulam structures. These connections require strict quality control and so are required to be fabricated in a factory. These connections do not provide any ductility, but have very good appearance (Buchanan and Fairweather, 1993).

Two examples of typical glued portal frame connections are shown in Figure 3.6. Figure 3.6 (a) shows the cross-lapped portal frame joint frequently used by McIntosh Laminates of Auckland and Figure 3.6 (b) shows a mitred finger joint, which is commonly used in Europe (Buchanan and Fairweather, 1993).

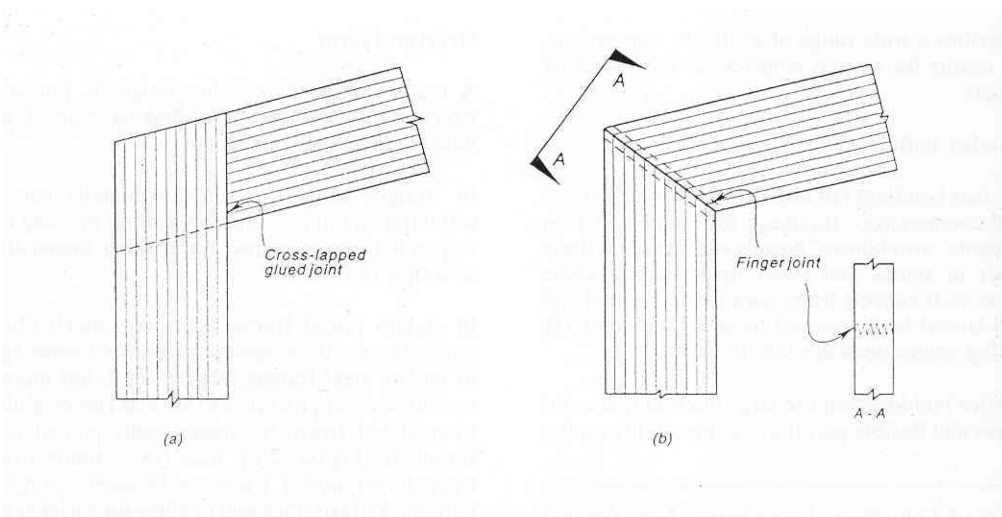
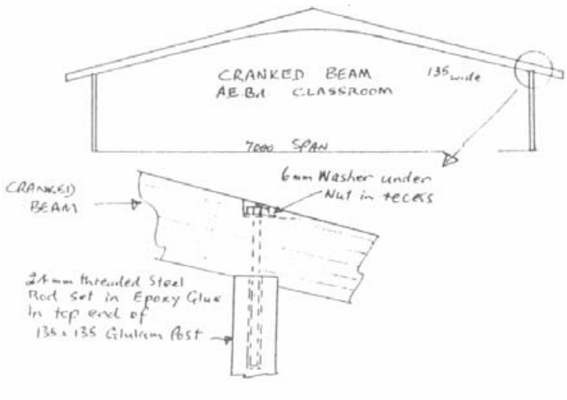


Figure 3.6 Glued portal frame connections (Buchanan and Fairweather, 1993)

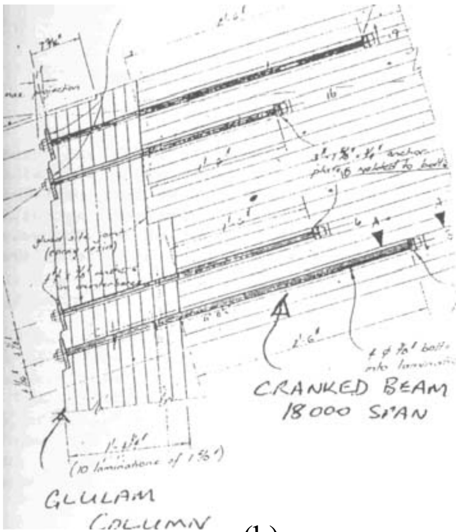
Design procedures for the cross-lapped joint are given by Buchanan (2002).

3.1.4 Epoxy-grouted steel rod connections

This research focuses on epoxy-grouted steel rod connections. These have been in use in New Zealand since the 1960s (McIntosh, 1989). Some of the early examples can be found in educational buildings with glulam structures built in this period (McIntosh, 1989). Some of these early connections are shown in Figure 3.7.



(a)



(b)

Figure 3.7 (a) Auckland Education Board classroom detail and (b) Te Awamutu College gymnasium knee joint detail (Reproduced from McIntosh, 1989)

Epoxy-grouted steel rods are very strong when glued in parallel to the grain of the timber, but are not as strong when glued perpendicular to the grain, as is the case in the rafter of Figure 3.7 (a) (Buchanan, 2002). To overcome this problem a steel hub can be used to connect the members. The use of the hub allows for all of the steel rods to be grouted in parallel to the grain of the timber, giving the greatest strength. The steel hub also allows for ductility to be designed into the connection (Buchanan and Fairweather, 1993). A schematic of this detail is shown in Figure 3.1(c) and a structure built using this connection is shown in Figure 3.8.

Figure 3.8 Glulam portal frame with steel hub and epoxy-grouted rods

Epoxy-grouted rods (without the use of steel hubs) have the advantage of having no exposed steel. This is highly advantageous in corrosive environments such as swimming pool enclosures. Epoxy-grouted rod connections in timber structures have been used widely to avoid corrosion (Buchanan and Fletcher, 1989).

A more complex version of the steel hub design was used in the construction of the Sydney Olympic stadium (shown in Figure 3.9). This hub connects steel rods grouted into the endgrain of multiple heavy timber members to create a large open truss. Extensive testing of epoxy-grouted steel rod connections in glulam timber was performed for this project, as reported by Gaunt (1998).

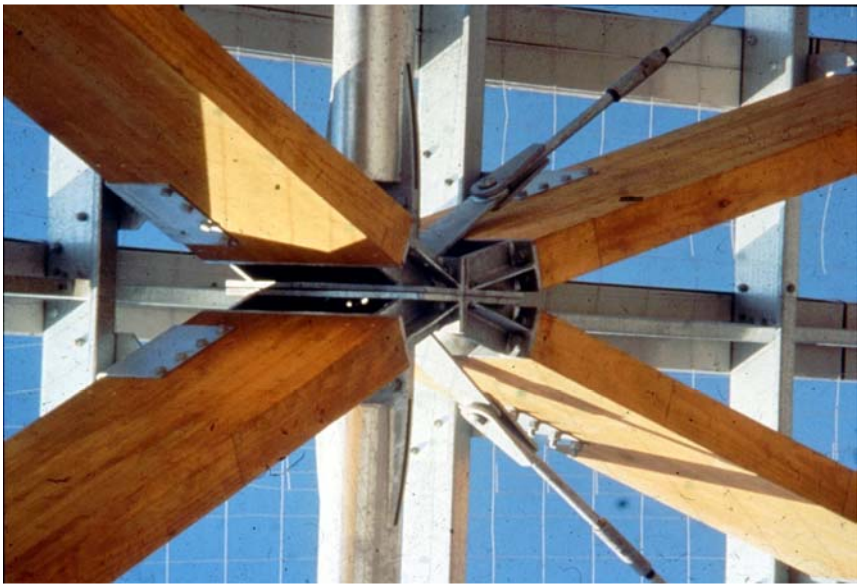


Figure 3.9 Steel hub connecting multiple glulam members of the Sydney Olympic stadium

More recent developments in LVL have seen the emergence of a moment resisting knee joint in LVL without the use of the steel connecting hub (van Houtte, 2003), as shown in Figure 3.10. This connection by van Houtte also provides a level of fire resistance to the steel. Having an unseen connection also improves the aesthetics and allows for clean lines along the structural members.

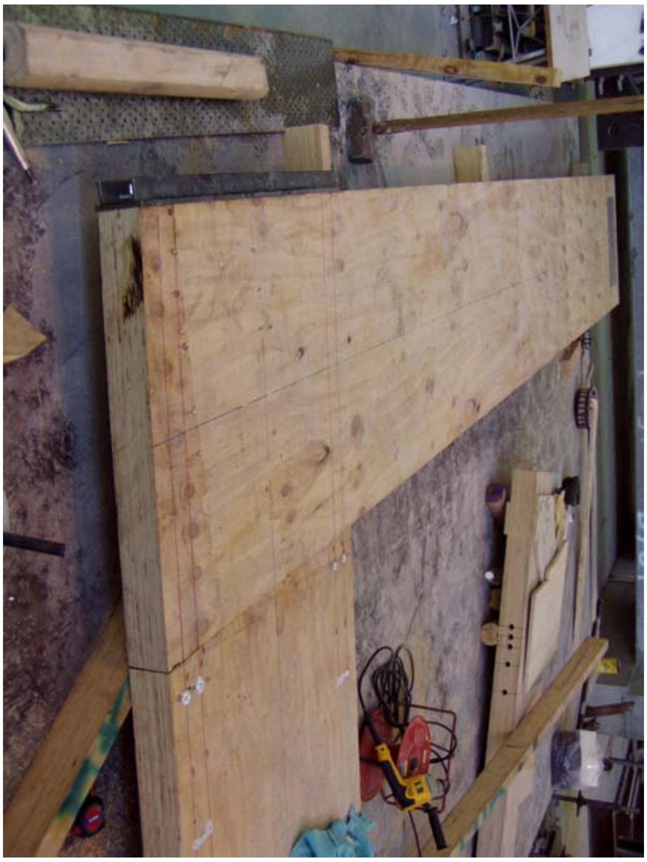


Figure 3.10 Epoxy-grouted steel rod connection in LVL timber (photo by A. van Houtte)

Design guidance for epoxy-grouted steel rod connections is given by Buchanan (2002). Specific details on the pull-out strengths for different adhesives, rods and species have been tested and details of these are given in 3.1.4.1.

3.1.4.1 Factors influencing strength of epoxy-grouted steel rods

Epoxy-grouted connections are dependent for their strength on the pullout loads of the grouted steel rods. These pull-out loads are dependent on many factors, such as the adhesive type, steel rod type, geometry of embedment and timber species.

Structural glulam and LVL in New Zealand are predominantly constructed from radiata pine, which removes variations in performance due to species. As timber is a naturally variable product there will still be some variation between pieces of timber.

Extensive research has been done to verify the impact of varying the depth of embedment, the diameter of the hole and the type of rod used, such as Deng (1997). Much of this work is summarised by (Buchanan and Moss, 1999) and is used in the design guidance given by Buchanan (2002).

In New Zealand the most widely used glues for grouted steel rod connections are epoxy resins, such as the West System (Gaunt, 1998), Araldite 2005 (Buchanan and Fletcher, 1989) and Nuplex K80 (Buchanan and Moss, 1999).

Adhesive type is important for grouted steel rod connections. Grouted steel rod connections are typically constructed using epoxy resin, but alternatives have been investigated. Kemmsies (2000) reported on tests using epoxy resins, polyurethane adhesives, sealant-adhesives and phenol-resorcinol formaldehyde. From these tests it was found that the sealant-adhesives were too flexible. This led to poor load carrying ability and unsatisfactory deformations. Two families of polyurethane adhesives were tested, one and two-pot varieties. Of these, the one-pot polyurethanes, like the sealant-adhesives, were found to have low strength and low stiffness. The phenol-resorcinol formaldehyde tested was modified to be gap filling, but was also found to have low stiffness. The two-pot polyurethanes and the epoxy resins tested were both found to have high strength and stiffness. The two-pot polyurethanes were found to be slightly weaker than the epoxy resins and were found to be prone to reacting with the moisture in the timber and creating bubbles in the adhesive, which reduced strength. Overall Kemmsies (2000) found the two-pot polyurethanes had similar characteristics to the epoxy resins tested. Creep and durability of the bond were not assessed in the study.

The steel rods used in epoxy-grouted steel rod connections must be strong enough to transmit the forces into the timber. As the epoxy-grouted steel rod connections are very strong this often requires the use of high strength steel rods (yield strength 680 MPa). Threaded rods have been found favourable over plain and deformed bars, as their profile allows for a better mechanical bond with the adhesive (Buchanan, 2002).

3.1.5 Dowelled connections

Another recent development in New Zealand in the field of connections in LVL timber includes the self-drilling dowel connection described by Scheibmair (2003). This connection involves connecting a steel plate between two pieces of LVL, with the steel plate being fixed using self-drilling steel dowels. This connection is illustrated in Figure 3.11. Timber connections using self-drilling steel dowels are used frequently in Europe and other parts of the world (Scheibmair, 2003). These provide hidden connections in heavy timber construction which are very strong (Larsen, 2003). Little development has been done in New Zealand for this connection.

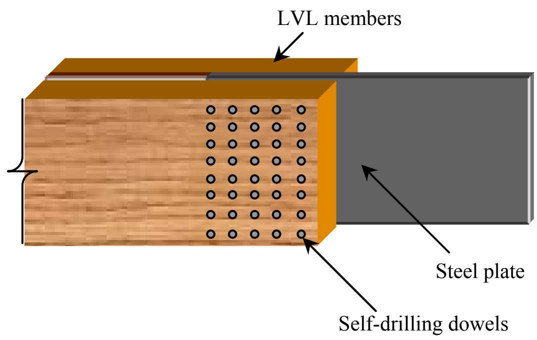


Figure 3.11 LVL connection using self-drilling steel dowels (Scheibmair, 2003)

The self-drilling steel dowel connection has the benefit of not requiring pre-drilling of holes for the dowels. This makes for easier construction due to the fact that there are no holes to line up, as there would be if bolts or a steel nail plate were used. These connections were shown to develop high strengths and good ductility in testing (Scheibmair, 2003).

Fire testing of this connection was not performed by Scheibmair. The current study had originally intended to investigate the issue of heat transfer through the steel components and in particular the char formed due to this transfer. It is thought that the heat transfer along the dowels may cause charring around the dowels. This could result in a loss of bond between the wood and the dowels, and a consequent loss of strength in the connection.

During this current research it was initially intended that these dowelled connections would be tested in the furnace specifically constructed for this project. Unfortunately irreparable damage to the custom-built furnace occurred during the phase three testing of the epoxy-grouted connections. This damage prevented any tests from being performed using these dowelled connections.

3.2 Fire performance

3.2.1 Background

While it is important to understand the fire resistance of structural connections, it is equally important to understand when structures are required to have fire resistance.

Under the New Zealand Building Code (BIA, 1992) building owners are not required to protect their own properties from damage by fire. They are, however, required to prevent fire spreading to neighbouring properties, and to prevent collapse of the building across the boundary. A building is also required to have sufficient fire resistance to ensure safe evacuation of the occupants and to allow for Fire Service intervention for rescue and fire fighting.

Methods of compliance with these requirements are offered by the Acceptable Solutions to the Building Code (BIA, 2001). Within the Acceptable Solutions, the fire resistance required of a structure depends upon its use, height and proximity to any boundaries. If a building is under single ownership, is remote from property boundaries and is a single level, then no fire resistance ratings are required. Once the building increases to two levels then a minimum of a 30 minute fire separation is required between each level. If the building is single level with a mezzanine floor then the mezzanine floor and its supporting structure are required to have a 15 minute fire resistance. Where a property is located near a property boundary the fire resistance rating depends upon the fuel load and ventilation conditions within the building, but this is typically 30 or 60 minutes.

From these requirements it can be seen that for a single storey building located remote from property boundaries no fire ratings would be required. Single storey portal frame buildings are often built using heavy timber construction and sometimes use epoxy-grouted steel dowel connections. Examples of some timber buildings with epoxy-grouted steel rod connections are described by Buchanan and Fletcher (1989). In these buildings fire rating requirements would be minimal, such as Jellie Park swimming pool (Christchurch), a single storey building remote from property boundaries. In this building there is a mezzanine floor and its supporting structure would be required to have a 15/15/15 fire resistance (BIA, 2001). The main portal frames would not be required to be fire resistant (BIA, 2001). Also, the Aquagym swimming pool (Christchurch) is single storey, but as one wall is on a property boundary this wall would be required to be fire resistant. Again, the main portal frames would not require a fire resistance rating, unless they are providing lateral support to the boundary walls.

Requirements by the building owners and insurers may increase the fire resistance of the structure above that required by the Acceptable Solutions (BIA, 2001). This may be done to provide a fire separation between tenancies, or to limit the damage to the building if a fire were to occur.

3.2.2 Fire resistance of timber and connections

All of the connections shown in section 3.1 have been proven through use to have suitable structural performance. It is also well known that heavy structural timber sections have good fire resistance and typically perform better than steel and concrete beams (Jackson and Dhir, 1996). Most of this performance can be attributed to timber's ability to char at a predictable rate, the insulative properties of that char and also that the timber tends not to deform until very near failure.

Research on timber at elevated temperatures has found that beneath the char layer there is a loss in strength of the remaining section. Green *et al.* (1999) found that at 50 °C the bending strength of timber can be reduced by between 10 and 25 % (depending on moisture content) and that the tensile strength can be reduced by approximately 4 %. Within a timber section exposed to fire temperatures there will be a zone beneath the char layer where the temperatures will exceed 50 °C. The strength loss in this zone must be included in any calculations of fire resistance.

Temperature profiles through a timber section are difficult to calculate easily and are best achieved using finite element numerical methods. For design purposes there are many simpler techniques for calculating the reduced strength of a timber section. These methods generally use a specified rate of char to predict the rate at which the outer layer burns away and then calculate the residual strength based on the remaining cross-section, with some reductions.

The various techniques for predicting the strength of a heavy timber beam when exposed to fire are described by Buchanan (2001). This includes the method used in New Zealand where for radiata pine a charring rate of 0.65 mm/min is used, with additional charring at the corners. This method ignores any reduction in strength due to heat affected timber beneath the char, with the residual section capacity being calculated using the full strength of timber beneath the char layer (Buchanan, 2002).

Other methods used include the effective cross-section method, where a layer of zero strength timber (typically 7 mm, but varies between codes) below the char layer is excluded from the strength calculation to account for the heat-affected timber below the char layer. Another alternative method to account for the loss in strength due to increased temperature is the reduced properties method. This method takes all of the residual section below the char layer as being at a reduced strength (Buchanan, 2001).

Where fire resistance is required, under the New Zealand loadings code (SNZ, 1992) the structure is required to carry a load lower than that for cold conditions. For ultimate limit state conditions a typical load combination is 1.2G & 1.6Q, where G is dead load and Q is live load. Under fire conditions the structure is required to carry only G & Q_u, where Q_u is the live load at ultimate limit state, which, for a roof, is zero. These lower loads must then be carried by the reduced section size calculated based on the anticipated duration of the fire, as described above.

It has been observed that the charring rate of solid timber is highly dependent upon the moisture content of the timber, the density and to a lesser extent the species (White and Dietenberger, 1999). For radiata pine the New Zealand standard (SNZ, 1993) gives this charring rate as 0.65 mm per minute, based on charring tests on radiata pine glulam (Buchanan, 2002). Research has shown that solid timber and glulam char at similar rates (Buchanan, 2002). Recent work by Lane *et al.* (2004) has shown that the radiata pine LVL used in this research project chars at 0.71 mm/min.

3.3 Fire resistance of connections

It is well known that a structure is only as strong as its weakest link. A timber beam can be designed to withstand major fire, but if the connections cannot maintain their strength at elevated temperatures then the structure will collapse.

3.3.1 Fire resistance of nailed connections

The fire resistance of nailed gusset plate connections relies predominantly on the fire performance of the gusset plate. Provided that the nails are long enough to remain embedded in solid timber it will typically be the loss of strength in a steel gusset plate, or the excessive charring of a timber gusset plate which will lead to collapse.

Tests on steel and plywood gusset plates were performed by Chinniah (1989) and are summarised by Buchanan and King (1991). Tests were performed on 5 mm steel gusset plates and 19 mm plywood gusset plates and used a variety of gypsum plaster boards and intumescent coatings for fire resistance. It was found that with similar levels of protection the plywood gusset plates had around twice the fire resistance of their steel counterparts (Chinniah, 1989). Using gypsum plaster board for protection of the connection it was found that the fire resistance of the connection was able to exceed that of the member (Buchanan and King, 1991).

An in-depth study into the performance of nailed connections was undertaken by Noren (1996). This study investigated the failure modes and load carrying capacity of nails when exposed to fire conditions. It was found that for load ratios of around 0.3 in unprotected nailed gusset connections that a 15 minute fire resistance could be achieved.

Fire resistance of punched nail plates is poor, as the metal is exposed and the teeth of the nail plates are not buried very deep into the timber. This means that the nail plate will heat very quickly and as the timber begins to char the nail plates will be quickly attached to only the char layer. Tests reported by Carling (1989) indicate fire resistance for a given nail plate of 8 minutes.

3.3.2 Fire resistance of bolted connections

Fire resistance of bolted connections requires external protection to prevent the bolts from being heated. This can be achieved by recessing the heads of the bolts to the depth of the expected char layer and then providing wood plugs to cover the bolt heads (Buchanan, 2001).

3.3.3 Fire resistance of glued connections

Glued connections will behave similarly as for solid timber, provided that the glue is able to withstand the high temperatures. Glued connections bonded with thermosetting adhesives such as resorcinol or melamine adhesives will behave well (Buchanan, 1999). If the connection is bonded with epoxy, or other thermoplastic adhesives it is likely the connection will lose strength at much lower temperatures.

3.3.4 Fire resistance of epoxy connections

Fire resistance of epoxy-grouted dowel connections is largely dependent upon two factors. One of these is the performance of the epoxy at elevated temperatures and the other is the insulative properties of the timber which is protecting the epoxy.

The insulative properties of the timber are a function of the thickness of the timber surrounding the connection. Thus, if additional insulation is required then it is a matter of increasing the thickness of timber surrounding the joint, or drilling the holes in a deeper part of the cross-section.

Temperature dependent behaviour of the epoxy is a function primarily of the type of adhesive being used. Epoxy resins are typically altered by changing the fillers added to them. Some of these fillers can improve handling characteristics and others can improve the high temperature performance (Harrison, 1986). One major drawback of fillers that increase performance at high temperatures is that they require curing at temperatures of around 120 – 150 °C.

One-pot epoxy systems are also available. Drawbacks of the one-pot systems are that they are heat cured and are unable to withstand sustained loading (Wilson, 1986). In epoxy-grouted steel rod connections there are substantial loads applied to the adhesive through the steel rods and the low strength of the one-pot epoxy makes it unsuitable.

Heat curing of epoxy is not feasible for grouted rod connections. Development of high temperature epoxies has focussed largely on the automotive industry where large ovens are used for curing paint finishes and can easily be used for curing epoxy (Harrison, 1986). Typically, for timber construction, the specimens can be up to 12 m in length (typical limit for transportation) and ovens of this size are not readily available. Heat curing would also not be possible if these joints were to be cast on site.

Very little research has been performed on the fire resistance of epoxy-grouted steel rod connections. Barber (1994) undertook a series of tests with steel rods grouted into glulam timber using West System epoxy resin heated in an oven and also in a standard furnace. These furnace tests indicated that that strength of the epoxy begins to reduce at 50 °C and that by 70 °C it had lost most of its strength.

Comparisons of the oven tests and the furnace tests by Barber (1994) found that for a given temperature in the furnace, a specimen heated in the oven could carry significantly higher load. This indicates that heating epoxy-grouted steel rod connections in a furnace under constant load is a more severe test than heating specimens in an oven and then removing them before subjecting them to a tensile test.

This disparity was also found by Spieth (2002), who performed similar tests to Barber, but using steel rods grouted into concrete. Neither author could provide a substantial argument for why this difference occurred.

3.4 Summary

Fire resistance of heavy timber sections is very good; however connections can be a weak point. Connections for these members can often contain epoxy-grouted steel rods as they perform well structurally. Epoxy does, however, lose strength at relatively low temperatures. The effect that this has on the epoxy-grouted connection in LVL timber will be investigated, with the intention of predicting its fire resistance.

4 Methodology

This section provides an overview of the materials used for the construction of test specimens, as well as the testing machines used in this research. Details of the preparation of the specimens are also given.

4.1 Description of LVL

The laminated veneer lumber (LVL) used in this project was supplied by Carter Holt Harvey (CHH) and was produced at their Marsden Point mill in Northland, New Zealand.

The LVL used for all of the cold and oven testing was 105 mm thick. This thickness is not typically available on the New Zealand market but was an export product destined for the Japanese market.

Although this product was not branded as HYSPAN (a brand used only in New Zealand) it was manufactured to the same specifications as HYSPAN (R. Davison, *pers. comm.*). In the furnace testing two thicknesses of LVL were used, the 105 mm thick material described above and also 63 mm thick HYSPAN, a typically available product on the New Zealand market.

Although LVL possesses a lower coefficient of variation than sawn timber (CHH, 2000), efforts were made to reduce variability. To achieve this the test specimens were cut from as few pieces of timber as possible. Design properties of the HYSPAN LVL used in this project are shown in Table 4.1. Some of these properties are the tabulated values for HYSPAN LVL and others were tested as part of this research.

Table 4.1 Design properties for HYSPAN LVL

Property	Symbol	Value	Data Source
Modulus of elasticity (MOE)	E	11.8 GPa	Test
Characteristic stresses			
Bending	f'_b	42 MPa	CHH, 2000
Tension parallel to grain	f'_t	27 MPa	CHH, 2000
Compression parallel to grain	f'_c	34 MPa	CHH, 2000
Shear in beams	f'_s	4.5 MPa	CHH, 2000
Compression perpendicular to grain	f'_p	12 MPa	CHH, 2000
Shear at joint details	f'_{sj}	4.6 MPa	CHH, 2000
Joint strength group		J4	CHH, 2000
Moisture content		9.5 – 10.5 %	Test
Density	ρ	590 - 620 kg/m^3	Test

HYSPAN LVL is constructed from 3 mm thick laminates of radiata pine, glued together with a phenolic adhesive to form a continuous billet approximately 1200 mm wide. This billet can be varied in thickness and can be purchased in lengths up to 12 m.

The phenolic adhesive used for bonding the laminations was not expected to lose strength during the testing at elevated temperatures (Pizzi, 1994). It was observed throughout the testing programme that there were no failures of the phenolic resin bonding the laminations.

4.2 Description of adhesives

Three adhesives were used for grouting the steel rods into the timber in this research. These are the West System epoxy resin, RE 500 epoxy resin and the hybrid HY 150 system.

4.2.1 West System epoxy

Designed as a boat building adhesive, the West System has been used successfully in research of epoxy-grouted dowels in the past (van Houtte (2003), Deng (1997), Barber (1994)) and also in construction of the Sydney Olympic Stadium (Gaunt, 1998).

This epoxy was supplied as a resin (ADR 310) and hardener (ADH 26) and was mixed by hand. To ensure that the 100:13 (by weight) resin to hardener ratio was followed accurately digital scales were used. All epoxy was mixed in small batches to ensure that it was able to be used within its gel time.

West System has low viscosity when mixed. For injecting into the specimens the epoxy was mixed and then poured into caulking tubes. These tubes could then be loaded into a caulking gun to enable injection into the test specimens.

Heat distortion temperature (HDT), also known as the glass transition temperature, was a critical characteristic of epoxy resins when evaluating performance at elevated temperatures. West System is specified as having a heat distortion temperature of 60 °C (Adhesive Technologies, 1992). This is the temperature at which the epoxy transitions from behaving a solid to behaving as a rubbery liquid. The HDT of an epoxy is measured by immersing a solid bar of cured epoxy into an oil bath. Supports are provided at each end of the test specimen and a weight is placed at midspan. The temperature is increased in the oil bath at a rate of 2 °C until the midspan deflection in the bar reaches 0.25 mm. The temperature at which this deflection occurs is the HDT of the epoxy (Lee and Neville, 1957).

4.2.2 RE 500 epoxy resin

Produced by Hilti Adhesives, the RE 500 is primarily intended for use where high strength is required for grouting steel bars into concrete. This product was supplied in a two-pack cartridge and dispensed using the Hilti MD 2000 dispensing gun. This gun was designed to hold the cartridge and feed both components into the nozzle at the appropriate ratios. Within the nozzle there is a spiral to mix the two components together so that the adhesive emerging from the nozzle is fully mixed.

RE 500 does not have a specified HDT. It does have a recommended maximum service temperature of 50 °C (D. Arandjelovic, *pers. comm.*). This service temperature is the HDT, with an unspecified safety factor. It was also stated that the full strength of the epoxy can be expected up to temperatures of 43 °C.

4.2.3 HY 150 adhesive

HY 150 is also produced by Hilti Adhesives and is intended for grouting steel rods into concrete where moderate strength and a fire rating are required. It was expected that being cement based HY 150 would not bond to timber as well as the epoxy resins. This product was included in this research because of its good performance under fire conditions.

HY 150 is formulated as a hybrid of urethane methacrylate and cement. Like the RE 500 it is sold in a 2 pack cartridge and is dispensed using the MD 2000 gun.

In concrete the HY 150 has a lower design strength than the RE 500, so it was expected that the ultimate loads achieved in timber using the HY 150 would not be as great as those for the RE 500. This was supported by the design data. The design data stated that the design tensile load for a 16 mm reinforcing bar, grouted 170 mm into concrete, was tabulated as 78 kN for RE 500 and 13 kN for the HY 150 (Hilti, 2002).

As with the RE 500, the HY 150 does not have a specified HDT, only a recommended maximum service temperature. For HY 150 this recommended maximum is 60 °C (D. Arandjelovic, *pers. comm.*).

4.3 Description of steel rods

During all of the testing the rods used were M16 threaded steel rods in two grades. Grade 8.8 steel rods were used for the test specimens with higher expected failure loads and mild steel rods were used for those with lower expected failure loads.

Grade 8.8 steel rods have a tabulated yield stress of 680 MPa and ultimate stress of 830 MPa. This is compared with a tabulated yield stress of 300 MPa and ultimate stress of 350 MPa for the mild steel bars. This gives a theoretical yield load of 137 kN for the M16 grade 8.8 bars and 60 kN for the M16 mild steel bars.

All of the threaded rods used in this testing were purchased from Blacks Fasteners of Christchurch. They were purchased in 1 m lengths and cut to length in the workshop. The rods were all zinc plated. No surface preparation was undertaken to improve the bond with the adhesives, as it is unlikely that this could be done reliably during construction of a building, and it is not necessary due to the excellent mechanical bond between the epoxy and the threaded bars.

4.4 Preparation of test specimens

All test specimens used in this research were produced from sheets of either 105, or 63 mm thick HYSPAN LVL. A table saw was used to cut the test specimens to the required widths and lengths.

Boring of the embedment holes was conducted on a metal-working lathe. Depending on the test, either a 20, 24 or 28 mm hole was bored, to a depth of either 300 or 395 mm.

The test specimens were clamped to a piece of steel angle section fixed to the tool post and the auger drill bit was placed in the chuck, as shown in Figure 4.1. Using the lathe to bore the holes allowed greater precision in the location, angle and depth of the holes. Holes were drilled with standard wood-drilling auger bits, which allowed holes up to 395 mm deep to be bored.

Figure 4.1 Lathe used for drilling test specimens, with drill bit in chuck and test specimen clamped to tool post.

4.4.1 Screwing of test specimens

For test specimens where screws were included to prevent splitting between the laminations, these were placed as shown in Figure 4.2. Screws used for this purpose were 14 gauge countersunk self-tapping screws, 100 mm in length. These screws were threaded over the entire length of their shank. These screws were placed either side of the grouted steel rod, as shown in Figure 4.2, with 8 screws used per test specimen.

4.4.2 Gluing of test specimens

Prior to the gluing, the holes in the test specimens were cleaned of loose debris using compressed air, to ensure that the adhesive was able to bond to solid timber.

For test specimens cast using the West System epoxy resin, the rods were set in place and the resin was filled through the side of the test specimen. This required additional holes to be drilled in the specimen. Resin was injected through the filler hole (refer Figure 4.2) until it was observed to be coming out through the breather hole.

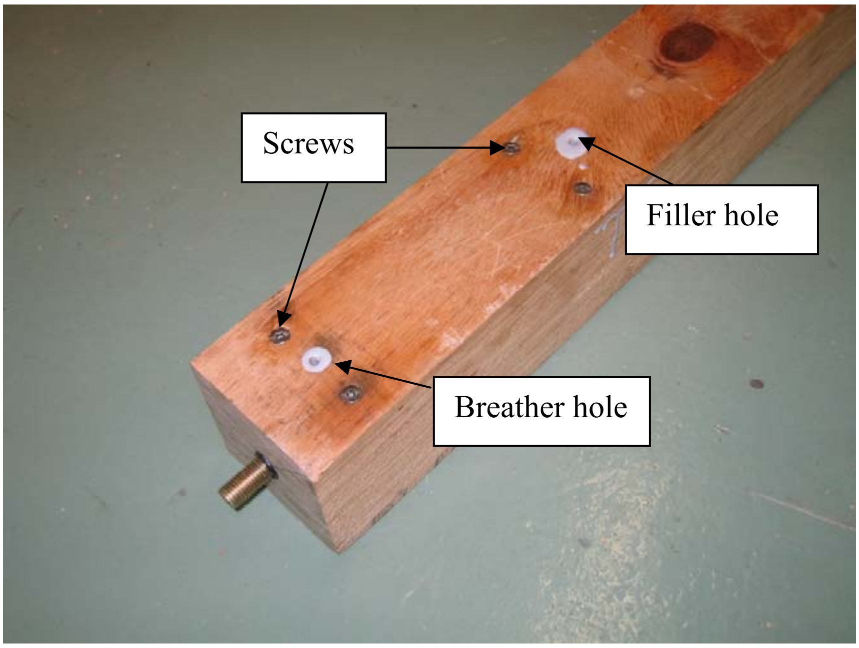


Figure 4.2 Test specimen cast using West System, showing screws, filler and breather holes.

To keep the rods centralised in the holes while the resin was curing, thin wire was wrapped around the rod near the tip and also near the beginning of the hole, as shown on the right in Figure 4.3. Leaks were prevented by sealing around the opening of the hole with flexible putty, which could be removed after curing was complete. Protection of the thread from leaked epoxy was provided by wrapping the exposed portion of the rod with insulation tape.

Figure 4.3 Showing the tie wire and nuts used for centring the rods in the holes

For some of the HY 150 specimens larger, 28 mm diameter holes were used with 16 mm rods. In these test specimens the rods were centred in the holes by the use of half of an M16 nut, as shown on the left of Figure 4.3. The nuts were split in half to reduce their volume and hence length of unbonded steel rod.

Test specimens using the Hilti adhesives (RE 500 and HY 150) were cast by injecting the adhesive into the embedment hole and then inserting the rod. This was the recommended practice for these adhesives (Hilti, 2002) and it was possible because the higher viscosity of the Hilti adhesives (when compared to West System) prevents the resin from running out of the hole.

With the Hilti adhesives it was not necessary to use wire to centre the rods in the holes (Hilti, 2002). It was found that by spinning the rod slightly during insertion it stayed aligned in the centre of the hole. Again, insulation tape was used to protect the exposed threads of the rods from epoxy overflow.

Problems were identified during the oven testing with the RE 500 adhesive. It was found that air bubbles were being trapped within the RE 500 adhesive. This problem was overcome by using an attachment to the injection nozzle. This is discussed in further detail in section 4.4.2.1.

All test specimens were cured in ambient laboratory conditions, without additional heating. Typically, during the period of gluing the temperature varied from 15 – 20 °C. All test specimens were left for a minimum of 24 hours prior to testing, as this allowed for complete curing of the adhesives.

Test specimens were all prepared with approximately 50 mm of steel rod exposed. When tested they had additional threaded rod coupled to these with threaded couplers, machined from 50 mm diameter steel bars. These couplers provided the load capacity required to transmit the load into the test specimens.

4.4.2.1 Removal of air bubbles from RE 500

Gluing of the RE 500 specimens required a special nozzle to prevent air bubbles being included in the cured resin. In specimens glued without this nozzle sometimes air bubbles were trapped, in these areas there was no adhesive to transfer load from the steel rod into the timber. Air bubbles were undetectable in completed test specimens prior to testing.

Upon failure of test specimens with air bubbles, these bubbles were clearly visible, as seen in Figure 4.4. In some specimens the rods were fully removed from the test specimen for inspection and the air bubbles can be clearly, as in Figure 4.5.

Figure 4.4 RE 500 specimen with air bubble visible after failure

Figure 4.5 Rod fully withdrawn from RE 500 specimen, with many air bubbles (seen as dark areas) in the resin

As the presence of air bubbles has a significant impact upon the ultimate load of the specimen it was very important that the problem be dealt with correctly and in a manner that provided consistent results.

When the Hilti resins are used in Europe for rods with long embedment a special nozzle is required (D. Arandjelovic, pers. comm.) to prevent air being mixed into the resin during gluing. Although these nozzles are mandatory in Europe, they are not available in New Zealand. To overcome the lack of availability a custom-made solution was prepared as shown in Figure 4.6. This nozzle consists of the standard injecting nozzle, with the supplied extension fitted. Fitted around the tip of the extension was a sleeve of plastic, which fitted tightly around the extension. The outer diameter was slightly smaller than the 20mm diameter of the hole in the timber.

Figure 4.6 Custom nozzle for preventing air bubbles in RE 500

When the hole is being filled with resin the nozzle is initially inserted to the bottom of the hole. As the resin is pumped into the hole it presses onto the sleeve and forces the nozzle back out of the timber. This leaves only resin (with no air bubbles) in the bottom of the hole. Once the required quantity of resin is injected the nozzle is removed and the rod is inserted. The pressure of the rod being forced in squeezes the resin towards the edge of the hole, but does not force air down into the hole.

This nozzle was found to be successful, with additional test specimens giving higher and more consistent ultimate loads. This method was used for most of the RE 500 samples in this research. Using this gluing method removed the air bubbles completely, as can be seen in a rod cast using this technique in Figure 4.7. It can be seen that none of the epoxy was visible as there was bonding to the timber over the complete length of the rod.

Figure 4.7 Fully removed RE 500 rod, without air bubbles in the resin

4.5 Testing equipment

All testing carried out in the three phases of this research involved tensile testing of specimens. Although the loading conditions were the same for all of the tests, the application of heat provided varying requirements for the testing equipment.

4.5.1 Instron testing machine

The cold tensile testing was performed in the Instron Testing Machine located in the Model Structures laboratory in the Civil Engineering Department at the University of Canterbury. This equipment has a capacity of 1000 kN, so was suited to the anticipated load range of 100 – 150 kN for these tests.

Load was applied at a constant rate of displacement by means of screw feeds. These screws, located within the vertical portions of the machine control the movement of the cross-head.

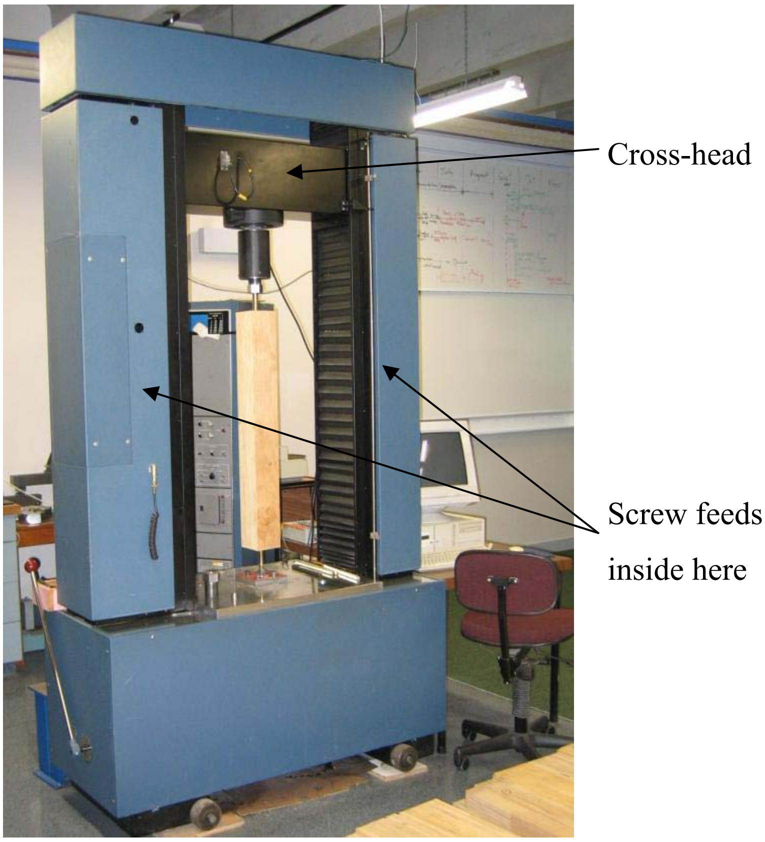


Figure 4.8 Instron testing machine, with test specimen loaded

Figure 4.8 shows the specimen, loaded in the vertical position ready for testing. The cylinder located above the test specimen is the load cell, which feeds directly to the data logger. There was also a potentiometer recording the elongation of the test specimen, fixed to the body of the test machine (obscured in Figure 4.8) to measure the movement of the cross-head during testing.

Calibration of the load cell was carried out using the Avery testing machine in the main structures lab at the University of Canterbury.

4.5.2 Custom-built testing frame

A testing frame was custom-built for use in this research and was used to perform tensile tests on timber specimens with grouted in steel rods. It was also fitted with a furnace for phase three of this research.

The custom test frame (without the furnace) was used during phase two of this research, where specimens were heated in an oven and then removed and tested (as seen in Figure 4.9). It was used during this phase of testing as it was able to be positioned much closer to the oven than the alternative testing machines, reducing the amount of cooling of heated specimens.

Figure 4.9 Custom-built test frame with specimen loaded

4.5.2.1 Strength of testing frame

The testing frame was designed to be able to break the strongest expected test specimen in this research. As all test specimens were constructed using 16 mm steel bars and the specimens cannot be stronger than the bar, the frame was designed to break a high strength 16 mm steel bar in tension.

The ultimate stress for grade 8.8 steel is 830 MPa. A 16 mm grade 8.8 steel bar has an ultimate load of 170 kN. The frame was designed with section sizes chosen to withstand the 170 kN tensile force without failing. The frame was not designed for deflection, as the deflection in these tests was not critical. Deflections of the frame were minimal; these were calculated and subtracted from the recorded elongations after completion of testing.

Details of the strength calculations for the frame can be found in appendix 1 to this report.

4.5.2.2 *Calibration*

It was critical to ensure that results achieved within the custom-built test frame were consistent with those achieved in the Instron machine.

Both the Instron and the custom-built testing frame measured the load on test specimens through a load cell attached to one end of the specimen. Both of the load cells for the testing frames were calibrated in the same Avery testing machine. Verification of this calibration and also of the loading regime between the Instron and the testing frame was conducted by means of replicating identical tests and checking for consistent results.

The Instron testing machine was used to conduct the cold testing of the test specimens and the custom-built frame was used for the oven and furnace components of this research. Used in this series of calibration tests were matched samples of West System resin, in a 105 x 105 mm test specimen, 900 mm long, with a 16 mm steel rod embedded 300 mm. Two of these specimens were tested in the Instron testing machine and two were tested in the custom-built testing frame. This was to ensure that not only the loads, but also the behaviours, such as deflection and failure modes, were consistent between the Instron and the testing frame.

The raw load deflection plots for the Instron testing machine and the custom testing frame are shown in Figure 4.10. It can be seen from this that although the ultimate loads were similar, that the deflections are different. This is due to two factors, the different rigidity of the Instron and the testing frame and also the length of the exposed steel rod over which the extensions were measured.

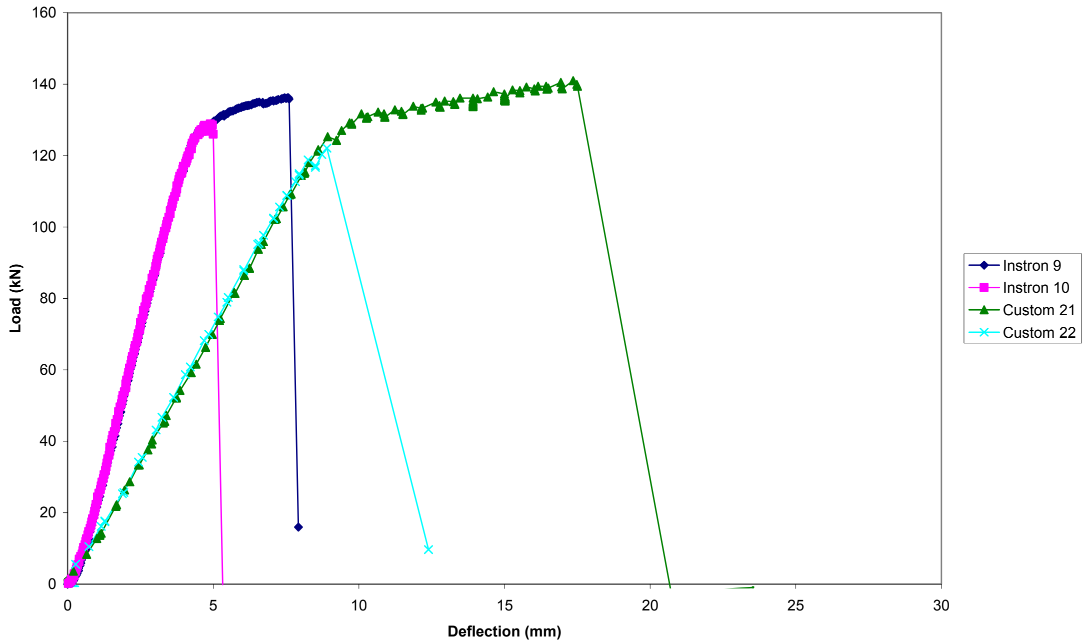


Figure 4.10 Load – deflection plot for test frame comparison, without corrections

It can be observed in Figure 4.10 that the recorded deflections in the custom built testing frame are much larger than in the Instron. This is due to the deflection in the reaction frame in the Instron being negligible, whereas the deflections of the custom-built testing frame amount to around 2.1 mm at 100 kN. Also critical was that in the custom-built testing frame the anchor points for the ends of the samples are further apart. This means that there was more threaded rod loaded and hence more elongation was recorded.

The deflections due to the additional steel rod and the deflections of the testing frame have been calculated and subtracted from those recorded in the custom-built testing frame. The corrected comparison is shown in Figure 4.11. It can be seen that the difference in deflection between the Instron and the custom-built testing frame amounts to approximately 0.3 mm at 100 kN. This difference is evident in Figure 4.11.

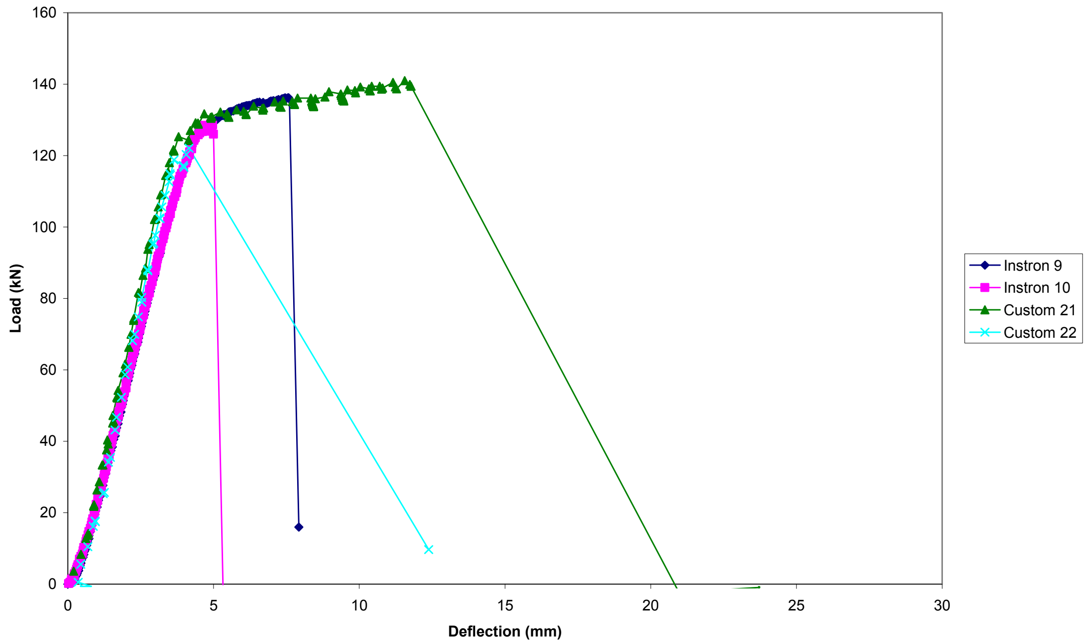


Figure 4.11 Loading comparison between Instron and custom-built test frame, including corrections for frame deflection and steel rod elongation

Details of these calculations for frame deflection can be found in appendix 2 to this report.

Overall, the loading curves between the Instron and the custom-built testing frame were seen to be consistent, providing that the displacement of the custom-built frame was accounted for.

4.5.3 Furnace

A custom-built testing frame with furnace was required for the furnace testing phase of this research. This testing frame was designed to allow a specimen to be held under a constant load while a heat flux was applied to the surface of the surface of the test specimen. This is the same testing frame used in the oven testing of the specimens, but with the addition of an electric furnace heater, as seen in Figure 4.12. During this phase of the research the test specimens were held under a constant tensile load while a heat flux was applied.

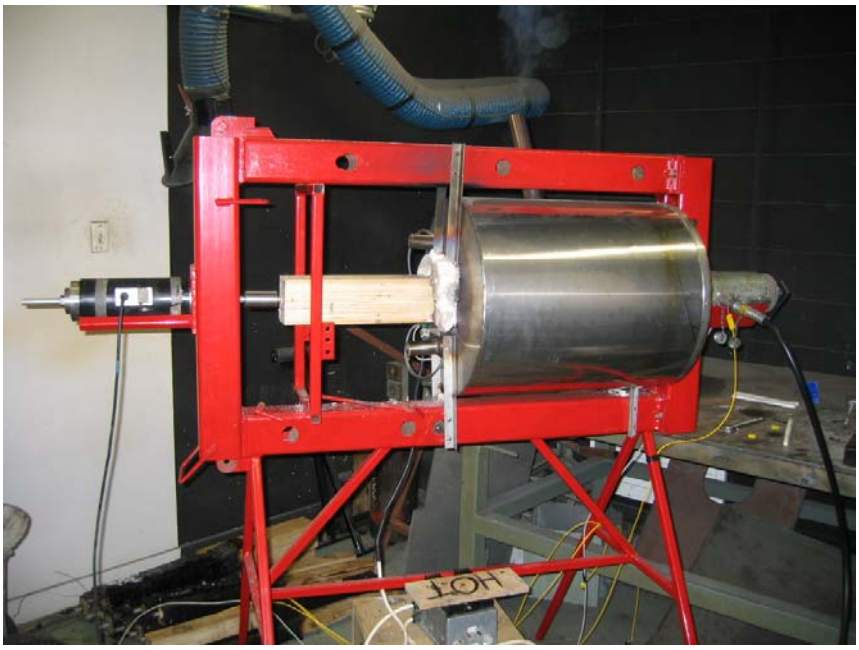


Figure 4.12 Custom-built testing frame, with furnace fitted and test specimen loaded

This cylindrical furnace was constructed from an outer stainless steel skin, with 100 mm of Kaowool mineral insulation and a stainless steel inner skin. An electrical element was fixed to the inner skin of the furnace. The element consisted of a 5.5 kW 3.65 mm diameter kanthal aluminium electrical coil, which was fixed to the inner skin in a spiral with porcelain insulators, as seen in Figure 4.13.

Figure 4.13 Electrical coil within furnace

As the coil consisted of a solid wire the surface of the element was electrically live, so it was not possible to attach a thermocouple as it would conduct current from the coil into the data logger. This prevented the element from being connected to a thermostat. Instead, the element was set at a constant level of electrical current, which gave a constant rate of energy into the system during the testing. Consequently, the test specimens were not exposed to a constant heat flux as they would have been if heated in a cone calorimeter.

4.5.3.1 Load applied during furnace tests

The load applied to the specimens during testing was intended to be indicative of the design load for this connection under fire conditions. Under the New Zealand Loadings Code (SNZ, 1992) a connection within a timber portal frame would typically be designed for the most severe loading combination of 1.2G & 1.6Q or 1.2G & S, where G is gravity load, Q is live load and S is snow load. As timber portal frame structures are lightweight the earthquake case is typically not critical.

For stability during fire the connections are be required to withstand the lesser load combination of G & Q_u, where Q_u is the ultimate limit state live load (zero for roofs). The actual values of G, Q and S are dependent upon the design of the structure and its location. Typically the ratio of the fire design loads to cold design case is around 0.3.

The design strength of the connection is not the average of the test results, but is a dependable strength of the connection. Typically this would be a lower fifth percentile value. As the testing involved in this research did not contain enough tests to accurately construct a distribution, the design strength has been taken to be 70% of the average failure load at ambient temperature.

The average cold failure loads for each of the three adhesives are shown in Table 4.2. As discussed above, the design strength was 70% of the average failure load and the test load was 30% of the average design load.

Table 4.2 Test loads for the furnace testing

Adhesive	Bar Grade	Average Failure Load (KN)	Design Strength (KN)	30% of Design (KN)	Test Load (KN)
RE 500	8.8	129	90.3	27.1	27
West System	8.8	131	91.7	27.5	27
HY 150	Mild	40	28	8.4	8

It can be seen in Table 4.2 that the 30% values for the two epoxy resins are similar. To allow for more accurate comparisons of data between the two resins the test load was chosen as 27KN for both types of epoxy. All loads were rounded to the nearest whole number to make application of the load easier.

During testing the load did reduce, partially due to a slight leak in the seals of the pump and partly as the test specimen was extending. As the test progressed the load was constantly monitored and adjusted. Typically the loads did not exceed ± 0.5 kN during the test until the specimen began to fail.

4.5.3.2 Removal of char from test specimens

Following the conclusion of testing, the layer of char was scraped from the test specimens to reveal the solid timber beneath. The dimensions of the remaining solid timber were recorded for each test specimen then from the original dimensions were deducted to give the depth of char.

This data was used to calculate the equivalent exposure to the ISO 834 heating regime, as explained in section 7.7.

5 Cold testing

Cold testing was carried out to compare the performance of the different adhesives under normal operating temperatures.

Previous tests by van Houtte (2003), Deng (1997), Barber (1994) and others have used the West System epoxy, but neither of the Hilti adhesives. As the current study was evaluating the effectiveness of Hilti adhesives it was necessary to determine their level of performance at ambient temperatures prior to evaluating their strength at elevated temperatures.

5.1 Assembly of specimens

The test specimens used for this testing were constructed of HYSPAN LVL with a 105 x 105 mm cross-section and a length of 900 mm. All the test specimens had the grain running in a longitudinal direction. Grade 8.8 (f_y = 680MPa) threaded steel rods were grouted into these holes using one of the three adhesives; HY 150, RE 500, or West System. Both ends of each specimen were grouted with the same adhesive. Detailed assembly of the test specimens is described in 4.4.

The base case for the cold tests was a 16mm diameter threaded steel rod, embedded 300mm in a 20mm diameter hole. This was based on the work by van Houtte (2003) where he found that this geometry gave the best results for testing parallel to the grain in LVL timber using epoxy resin.

5.2 Results

The failure observed in all of the tests in this series was brittle. Failures of all of the specimens were observed to be either in the timber, or at the wood/glue interface. There were no failures of the adhesive or steel rod. Ultimate loads and failure modes for each of the initial test specimens are shown in Table 5.1.

Table 5.1 Comparison of results for cold test specimens

Sample	Adhesive	Embedment length (mm)	Hole diameter (mm)	Screws	Ultimate load (kN)	Failure mode
1	RE 500	300	20	-	97	1
2	RE 500	300	20	-	120	1
3	HY 150	300	20	-	35	2
4	HY 150	300	20	-	45	2
5	RE 500	300	20	In wood	131	3
6	RE 500	300	20	In wood	134	3
9	West	300	20	In wood	136	3
10	West	300	20	In wood	126	3

Failure modes for all of the test specimens are discussed in section 5.3.

The results indicate that for epoxy specimens (RE 500 and West System) where screws were placed perpendicular to the laminations that the ultimate strengths were similar. Failure modes for these test specimens were also the same, indicating that the behaviour was the same for both adhesives.

These results show that there was a significant difference in strength between the epoxy specimens and those grouted with HY 150. This difference was in the order of a factor of four. These findings suggest that HY 150 is unsuitable as a direct replacement for epoxy resin for timber connections with grouted steel rods.

5.3 Failure modes

Three distinct failure modes were observed during the cold testing of the specimens, as noted in Table 5.1.

5.3.1 Mode 1 failure

Mode 1 failure was observed in the first tests of the RE 500 specimens. Mode 1 failures involved a splitting of the LVL parallel to the glue lines. In conjunction with this splitting the rod pulled out a portion of the timber the full width of the section (as seen in Figure 5.1).

Although this failure mode only occurred in the RE 500 specimens it is expected that it would have occurred had the West System been tested without screws perpendicular to the lamination, as throughout the testing programme the two epoxies behaved in a similar manner.

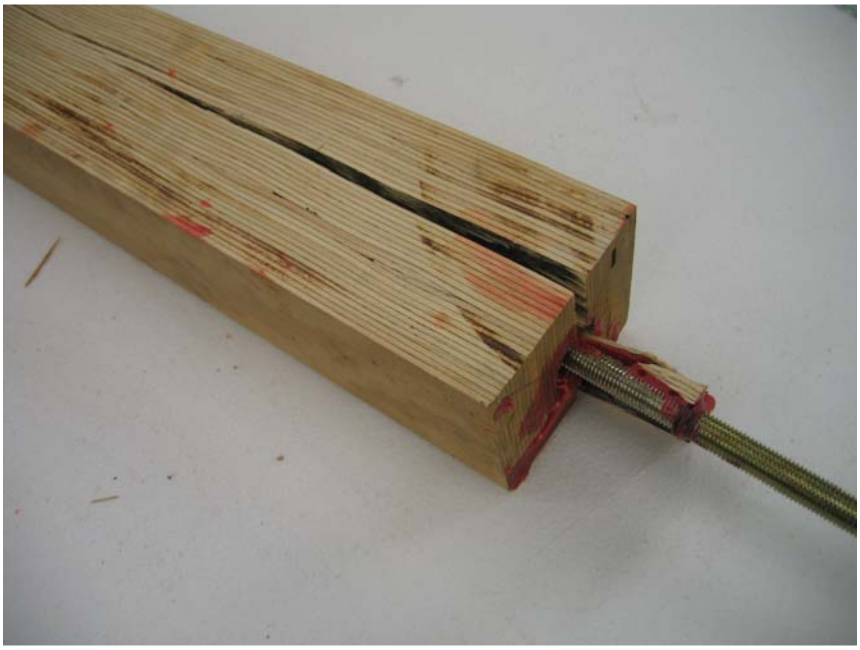


Figure 5.1 Mode 1 failure in RE 500 specimen

Mode 1 failures suggest that there was sufficient bond both between the adhesive and the rod and between the adhesive and the timber, but that extracting the rod created a splitting force perpendicular to the laminations. This splitting force was consistent with results observed by van Houtte (2003) in testing of similar specimens.

Similar observations to this current research and to van Houtte (2003) were recorded by Buchanan and Moss (1999) in glulam timber. Buchanan and Moss (1999) reported tests of M20 bars epoxy-grouted into glulam specimens with smaller, M4 rods grouted perpendicular to the laminations of a specimen for reinforcement. They found that the reinforcement increased the tensile strength sufficiently to cause the failure to occur at the anchorage end of the specimen. The anchorage end had twice the embedment depth, but did not have the transverse reinforcing rods (Buchanan & Moss, 1999).

To prevent mode 1 failures van Houtte (2003) fixed self-tapping screws perpendicular to the laminates. These screws then act in tension to keep the laminates from splitting apart and improve the ultimate load of the connection. Based on van Houtte's research, self-tapping screws were therefore used in the remaining epoxy-grouted specimens in the current study. The resulting strengths and failure modes are recorded in Table 5.1.

In van Houtte's research (2003) he also observed splitting perpendicular to the laminations. This was observed by van Houtte (2003) in test specimens with screws perpendicular to the laminations and when the edge distance in the plane of the laminations was greater than the edge distance perpendicular to the laminations. This failure mode with cracking across the laminations was not observed during the cold testing phase of the current research. It was thought to be due to all test specimens tested in this phase of research having equal edge distances both perpendicular and parallel to the laminations. It was however, observed in some of the furnace tests of the current research and was classed as a mode 7 failure.

5.3.2 Mode 2 failure

With the hybrid HY150 adhesive system the behaviour was very different to the epoxies, with significantly lower ultimate strengths. Failures for the HY150 specimens occurred consistently at the glue/wood interface, with insignificant amounts of wood being attached to the glue after failure. This failure mode was in direct contrast to that of the epoxies where the failures were within the wood, with significant amounts of timber being removed with the rod.

Mode 2 failures occurred only in the HY 150 tests and was the only failure mode found in tests of the HY 150 system. Figure 5.2 shows this failure mode, with the withdrawn core of adhesive clearly visible.

Figure 5.2 Mode 2 failure, typical for HY 150

Due to the failure of the HY 150 specimens occurring at the wood/glue interface, there is concern that the failure may lead to durability problems (H. Bier, pers. comm.). As the failure is at the surface of the glue it demonstrates that the HY 150 did not penetrate into the timber. Without this penetration the strength of the bond is highly influenced by changes in the surface of the timber to which it is connected. Changes such as swelling due to moisture fluctuations could decrease the bond strength and lead to premature failure.

Under the New Zealand Building Code (BIA, 1992) a connection such as this would be required to have a service life of not less than 50 years. If it cannot be guaranteed that it would survive the environmental conditions imposed upon it for 50 years then it would not be suitable for use. As the durability of the connection is dubious then it would not meet this requirement without further verification and possible modification. If it cannot meet this requirement then this adhesive is not able to be used in this form of structural connection.

5.3.3 Mode 3 failure

Mode 3 failures occurred when the epoxy specimens had screws holding their laminations together effectively. Mode 3 failures involved the steel rod pulling out of the test specimen and taking with it a significant amount of timber. In this instance the timber was found to fail both across and along glue lines and roughly formed an annulus of timber surrounding the steel rod, with a thickness of approximately 5 – 15 mm. An example of a mode 3 failure can be seen clearly in Figure 5.3.

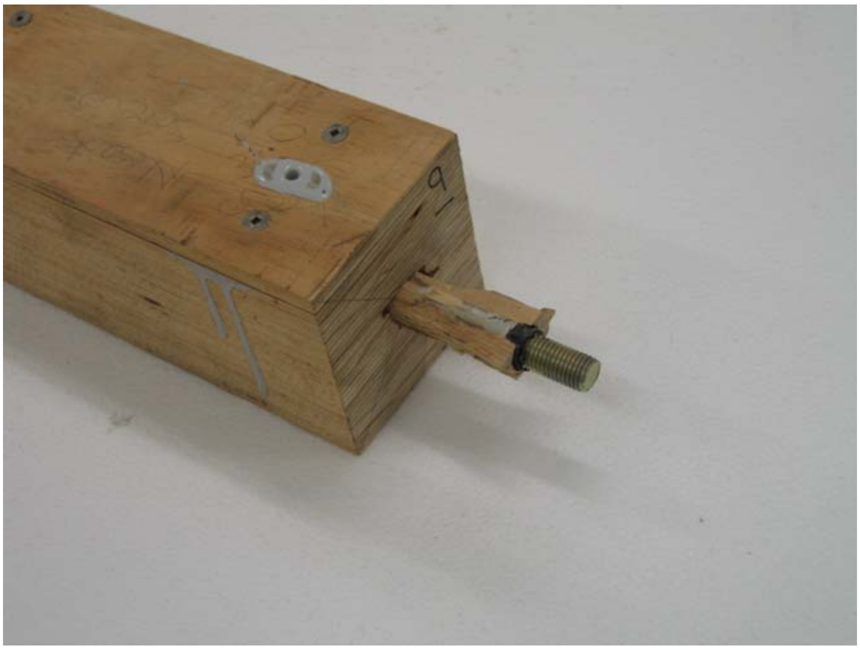


Figure 5.3 Mode 3 failure in West System specimen

5.4 Variations to HY 150 system

Variations were made to the HY 150 configuration originally tested, as it was found from test specimens 3 and 4 that the pullout strength was considerably lower than that recorded for epoxy resin. Both specimens 3 and 4 failed at the wood/glue interface and it was hypothesised that, either if some mechanical bond could be achieved at this interface, or if the surface area of this interface could be increased then the strength of the connection would be increased.

These variations included increasing the embedment depth, increasing the diameter of the embedment hole and also putting screws through the adhesive to provide a mechanical connection between the adhesive and the wood. Further testing was then conducted to test these hypotheses.

5.4.1 Increasing surface area

Failure in specimens 3 and 4 failure occurred at the wood/glue interface. From this it was further hypothesised that there must be a limiting shear stress at which these failures were occurring. Based on this theory it was postulated that if the thickness of glue surrounding the rod was increased then the ultimate load of the connection should also increase.

Shear stress along a surface can be calculated using Equation 5.1. In specimens 3 and 4 the steel rod was embedded 300 mm in a 20 mm diameter hole. Ultimate loads for these test specimens were 35 and 45 kN, giving shear stresses at failure of 1.86 and 2.39 MPa.

$$Stress = \frac{Force}{Area}$$ **Equation 5.1**

Tests conducted using epoxy resins had an average pullout load of approximately 130 kN. To achieve this load with HY 150 and using 2.10 MPa as the shear stress limit, using Equation 5.1 the required surface area of the embedment hole is 61.9 x 10^3 mm^2. This was compared to a surface area of 18.8 x 10^3 mm^2 for specimens 3 and 4. Using this theory, to achieve an ultimate load of 130 kN with HY 150 would require a 65 mm diameter hole 300 mm deep, or a 50 mm diameter hole 395 mm deep (the maximum depth achievable with a standard auger bit). Holes of these diameters however are impractical, as boring a 50, or 65 mm diameter hole to these depths could not be achieved using a standard auger or spade bit.

To confirm that this theory of a limiting shear stress was accurate the embedment depth and diameter were increased, although not to the extent calculated above. Two test specimens were tested with an embedment hole 20 mm in diameter and 395 mm deep and a further two specimens were tested with a 28 mm embedment hole, 395 deep. Pull-out loads and shear stresses at failure are recorded in Table 5.2. Failure modes for all of these specimens were as for the original HY 150 tests, with failure at the wood/glue interface.

Table 5.2 Comparison of shear stress with varying hole geometry

Hole diameter (mm)	Hole depth (mm)	Surface area (mm^2)	Test	Ultimate load (kN)	Shear stress (MPa)	Average shear stress (MPa)
20	300	18.8×10^3	1	35	1.86	2.13
			2	45	2.39	
20	395	24.8×10^3	1	46	1.85	1.95
			2	51	2.05	
28	395	34.7×10^3	1	54	1.55	1.60
			2	57	1.64	

From Table 5.2 it can be seen that by increasing depth the limiting shear stress remains fairly constant and the pull-out loads increase as expected. However, by increasing hole diameter the shear stress at failure decreases and the expected increase in pullout strength is not gained.

A possible explanation as to why the shear stress at failure decreased is that there could be some shrinkage of the HY 150 during curing. According to Hilti (2002) HY 150 has negligible shrinkage during curing. It may be that when this product is applied with a 1 or 2mm thickness around the rod, as specified by Hilti (2002), the shrinkage may be negligible, but when tested at 6mm thickness the shrinkage becomes noticeable. If slight shrinkage of the adhesive did occur during curing it would pull away from the edge of the hole and decrease the bond.

An implication of these results is that the shear stress at failure for the HY 150 specimens is not independent of embedment depth and diameter, so increasing the hole diameter and embedment depth do not yield the increased ultimate loads which were expected. This means that another method of increasing the ultimate load is required if the HY 150 system is to perform similarly to the West System and RE 500 epoxy systems.

5.4.2 Adding a mechanical bond

Another technique tested to increase the ultimate load of HY 150 was to create a mechanical connection between the adhesive and the inner surface of the wood in the hole. In a concrete specimen this would typically be achieved by roughening the surface of the hole (Hilti, 2002). The manufacturers of HY 150 require the inner surface of the hole be roughened if it is bored into concrete using a diamond drill (Hilti, 2002).

In timber it is not feasible to roughen the inner surface of the hole, as the roughening process breaks the fibres of the timber, which decreases its strength. Eistetter (1999) attempted to create a mechanical bond by roughening the inner surface of holes in timber prior to grouting with cementitious grout, but only achieved a marginal increase in bond strength.

Further attempts by Eistetter (1999) to create a mechanical bond used screws and bolts passing through the timber and the grout. These gave higher ultimate loads than for the roughening of the hole. Because the cement grout had very little adhesion to the timber the anchorage was principally through the screws or bolts, which led to unacceptably large deflections prior to failure.

As the HY 150 was found to give some adhesion in the current study, it was decided to attempt to create a mechanical bond using screws through the adhesive, with the hypothesis that the adhesive would prevent the displacement and the screws would provide an increase in ultimate load.

5.4.2.1 Configuration of test specimen

The test specimens were drilled with an embedment hole of 28mm diameter at each end, to a depth of 300mm. Four screws were fixed in each end of the test specimen, staggered either side of the steel rod, as shown in Figure 5.4.

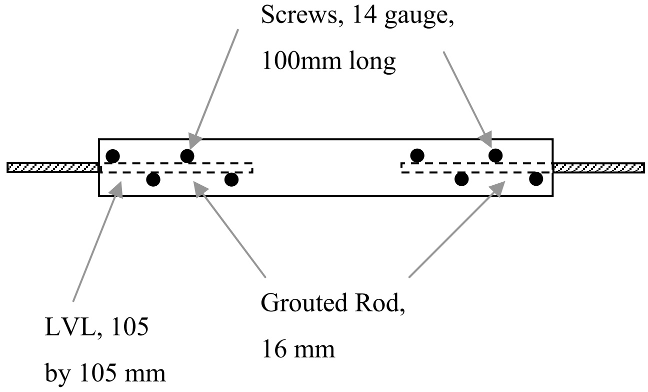


Figure 5.4 Configuration of HY 150 specimen with screws in adhesive

Screws used for the fastening were 14 gauge, 100mm long stainless steel, self-tapping, countersunk and threaded over their entire length. These were the same screws used for preventing splitting in the epoxy specimens.

5.4.2.2 Results

When tested the specimens initially behaved as if the screws were not present. As load was applied the same load-displacement curve was followed, indicating that there was no slip in the adhesive, followed by an initial failure at a similar load to the HY 150 specimens without screws. This behaviour can be seen in Figure 5.5, where all four of the test specimens had an initial failure at similar loads of around 45 kN. The load dropped suddenly due to loss of bond, and the subsequent increase in load is due to bending of the screws. Following this slight increase in load, another drop in load is noted as the screws fail in bending. This occurred at a load of approximately 30 kN.

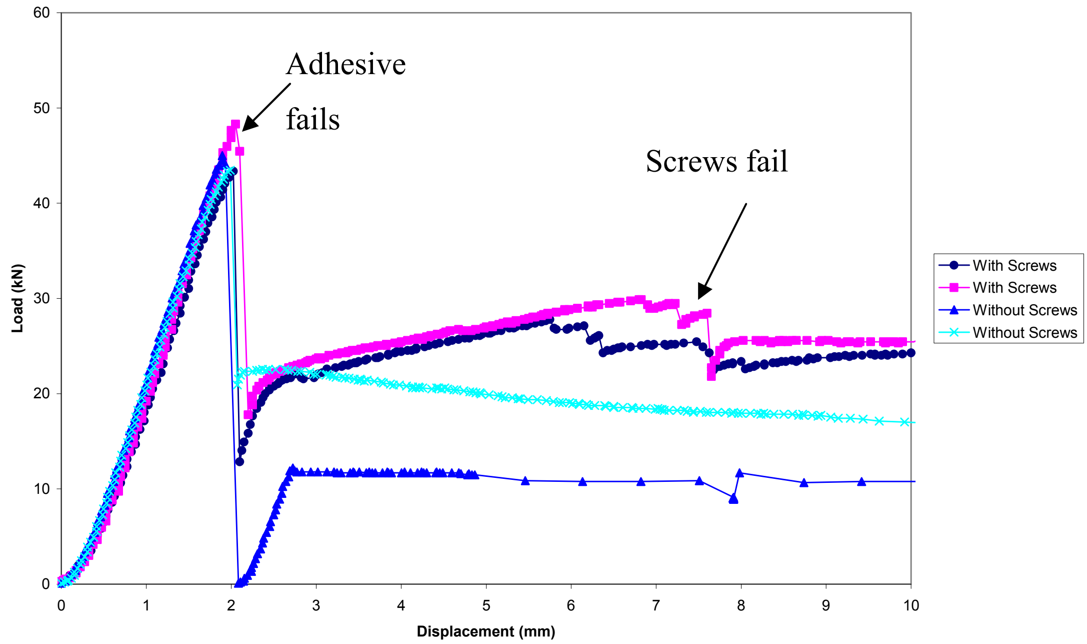


Figure 5.5 Comparison of effect of having screws in glue for HY 150 adhesive

This technique of using screws to increase ultimate failure load was not successful. It is predicted that this lack of effect was due to the differing rigidity of the screws and the glue. For the screws to begin carrying load there had to be some slip of the steel rod, but this only occurred when the brittle failure of the glue occurred and it stopped carrying load. To obtain any useful benefits from the screws, the load on the specimen would need to be distributed between the screws and the glue prior to any failure occurring and this did not occur as the screws were eventually unloaded at the time when the HY 150 bond failed.

5.4.3 Comparison of HY 150 improvements

Overall comparisons of the performance of the altered HY 150 system shows that although there was an improvement in the ultimate loads of the specimens when the hole diameter and embedment depth were increased, it was not as great as was anticipated. Similarly, the inclusion of screws in the adhesive did not make a significant difference to the overall performance of the connection.

A summary of the cold test results for the HY 150 system is shown in Table 5.3. It is worth noting that the failure mode in all specimens was at the wood/glue interface. This indicates that the strength of the connection never reached the strength of either the bar or the timber and hence was uneconomical as a method of grouting steel rods into timber sections.

Table 5.3 Comparison of HY 150 failure loads

Sample	Adhesive	Embedment length (mm)	Hole diameter (mm)	Screws	Ultimate load (kN)	Failure mode
3	HY 150	300	20	-	35	2
4	HY 150	300	20	-	45	2
7	HY 150	390	28	-	57	2
8	HY 150	390	28	-	54	2
11	HY 150	300	28	In glue	43	2
12	HY 150	300	28	In glue	50	2
13	HY 150	395	20	-	46	2
14	HY 150	395	20	-	51	2
15	HY 150	300	28	-	43	2
16	HY 150	300	28	-	45	2

5.5 Comparison with other testing

This work builds on previous research undertaken at the University of Canterbury using a similar methodology. Accordingly, similar results would be expected and thus it was important that comparisons be made with the previous work. It was expected that the results for the cold testing in this research would be similar to those of van Houtte (2003) and Deng (1997).

5.5.1 Van Houtte (2003)

The results from Van Houtte (2003) are the most directly comparable to the current research as his tests included 16mm steel rods embedded in LVL using West System epoxy. Van Houtte's research tested many combinations of embedment depth, rod diameters and edge distances, but with only single replications of each, which does not provide a good distribution of results.

From his work van Houtte developed an empirical equation for predicting the ultimate load of a single steel rod embedded in LVL parallel the grain, grouted with West System epoxy resin. This equation is shown in Equation 5.2. Van Houtte (2003) found this formula to be accurate for embedment depths between 50 and 400 mm.

$$F = K\left(\left(1.855x10^{-4}\right)h^2lf'_s + 15\right)$$ **Equation 5.2**

Where:

- F = ultimate load of connection in kN
- K = Reduction factor, equal to 0.85
- h = Embedment hole diameter in mm
- l = Embedment depth in mm
- f'_s = LVL shear stress in MPa

This equation includes a 0.85 reduction, or safety factor. As this equation is being used to compare predicted ultimate loads with actual loads this reduction factor will be taken as 1.0.

Comparisons using this equation can be seen in Figure 5.6. In these comparisons K was taken as 1.0, f'_s was taken as 4.5 MPa (from Table 4.1) and the hole diameter as 20 mm.

5.5.2 Deng (1997)

Research by Deng (1997) generated design guidance on the pullout loads for epoxy-grouted rods in glulam timber. Some of Deng's (1997) testing was performed using West System and although the timber was different the ultimate loads should have been fairly similar.

Deng's research allowed him to propose an equation to calculate the ultimate load of the connection under tensile load.

$$F = 10.94k_bk_ek_m\left(\frac{l}{d}\right)^{0.86}\left(\frac{d}{20}\right)^{1.62}\left(\frac{h}{d}\right)^{0.5}\left(\frac{e}{d}\right)^{0.5}$$ **Equation 5.3**

Where:

- F = ultimate load of connection in kN
- d = steel bar diameter in mm, valid for $16 \leq d \leq 24$
- l = embedment length in mm, valid for $5d \leq l \leq 15d$
- h = hole diameter in mm, valid for $1.15d \leq h \leq 1.4d$
- e = edge distance from centre of steel to edge in mm, valid for $1.5d \leq e \leq 3d$
- k_b = bar type factor (1.0 for threaded rod)

k_e = epoxy type factor (0.86 for West System)

k_m = moisture content factor (1.0 for < 18 %)

Predicted failure loads using Equation 5.3 can be seen in Figure 5.6, with predictions using van Houtte's formula and experimental data from the current research.

Experimental ultimate loads shown in Figure 5.6 were all from test data of 105 x 105 mm LVL specimens, with 16 mm threaded steel rods grouted into 20 mm diameter holes.

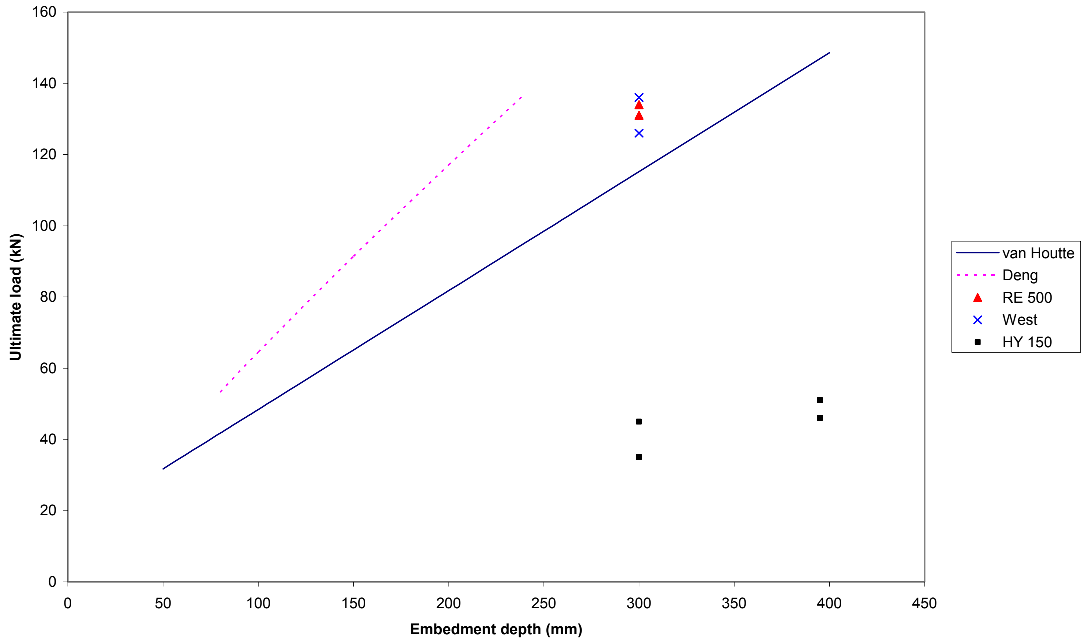


Figure 5.6 Comparison of predicted and experimental ultimate loads

It can be seen from Figure 5.6 that the experimental results for the two epoxy resins (RE 500 and West System) have higher ultimate loads than predicted by van Houtte but lower than those predicted by Deng. In his research van Houtte (2003) notes that his equation was based on a limited set of experimental results and that the correlation factor (1.855 x 10^{-4}) may not be entirely accurate. The experimental results shown in Figure 5.6 indicate that the correlation factor may be too low by around 15%.

It was expected that Deng's (1997) results may differ from those obtained in this research due to the Deng's formula being generated from testing in glulam and this research being conducted using LVL. The difference between the predicted results using Deng's (1997) equation (Equation 5.3) and the experimental results indicate that epoxy-grouted steel rods are stronger when grouted into glulam timber than when grouted into LVL. The difference may not be quite as great as that shown in Figure 5.6 as the embedment depths tested exceed the limits on embedment depth provided for Deng's equation (Equation 5.3).

5.6 Conclusions from cold testing

Both brands of epoxy resin showed similar performance under cold conditions, both in terms of failure modes and ultimate loads. This will allow for either resin to be used in the connection with the same design strength.

Ultimate loads for the HY 150 specimens are significantly lower than those for the epoxy resins, by approximately a factor of four. This will prevent the HY 150 being used as a substitute for the epoxy resins in this type of connection.

As the failures of the HY 150 specimens are consistently at the wood/glue interface there is also doubt as to the durability of this glue when bonded to timber. Attempts to increase the ultimate load for this adhesive by means of increasing the hole diameter and providing a mechanical bond with screws proved to be ineffective.

Comparisons with earlier work by Deng (1997) and van Houtte (2003) show that the cold results in this research are similar to those achieved previously and hence that the fire testing results achieved in the latter portion of this research can be applied to the previous work by these people.

6 Oven testing

Phase two of the testing for this research involved testing specimens at constant elevated temperatures. These tests were conducted to evaluate the strength loss of the adhesives with increases in temperature.

6.1 Setup of tests

Test specimens tested in this phase were the same as those used for the cold testing. Test specimens were assembled from 105 x 105 mm LVL, with 16 mm steel rods embedded 300 mm into each end of a 900 mm long specimen. The grades of the rods used for these tests were either mild or grade 8.8 steel, chosen to match the anticipated failure load. Mild steel was used in the tests where it wasn't expected to yield, as it was cheaper than grade 8.8 steel. The rod grades for each test are shown in Table 6.1.

Table 6.1 Testing regime for oven heated samples

Adhesive	Rod Grade	Test temperature
West System	Grade 8.8	50 °C
West System	Grade 8.8	75 °C
West System	Mild	100 °C
RE 500	Grade 8.8	50 °C
RE 500	Grade 8.8	75 °C
RE 500	Mild	100 °C
HY 150	Mild	50 °C
HY 150	Mild	75 °C
HY 150	Mild	100 °C

Three temperatures were tested, 50, 75 and 100 °C. These temperatures were chosen to give a spread of loads at a range of temperatures. Observations by Barber (1994) showed that West System epoxy lost a large portion of its strength by 100 °C. If the HY 150 and RE 500 adhesives were maintaining their strength at 100°C then further tests would have been performed at higher temperatures.

To achieve the temperatures required for this testing the test specimens were heated in a large oven.

6.1.1 Time in oven

To ensure that the time the specimens were left to heat in the oven was sufficient, an instrumented test specimen was heated and the temperatures of the oven and within the test specimen were recorded.

This test specimen was a standard 105 x 105 mm specimen, with steel rods epoxied into both ends. Within the epoxy at one end were three, type K, thermocouples, embedded 50, 150 and 300 mm into the test specimen. While the test specimen was in the oven there was also a type K thermocouple adjacent to the specimen to measure the air temperature within the oven.

An oven temperature of 100 °C was chosen for this test as it was the hottest of the oven heating tests and hence it would take the greatest length of time for a test specimen to achieve this temperature.

In planning this phase of testing it was anticipated that the specimens would be left in the oven overnight, prior to being strength tested. This was a duration of approximately 16 hours. While measuring the temperatures within the oven and specimen it was important that the oven was not opened as this would have resulted in an alteration in the conditions within the oven. To enable this to be achieved all of the thermocouple wires were fed through a gap between the front doors of the oven to allow the recordings to be made from outside the oven.

Temperature measurements were taken over a period of approximately 16 hours and the results are shown in Figure 6.1. The shallow, medium and deep temperatures refer to the 50, 150 and 300 mm deep thermocouples within the epoxy resin.

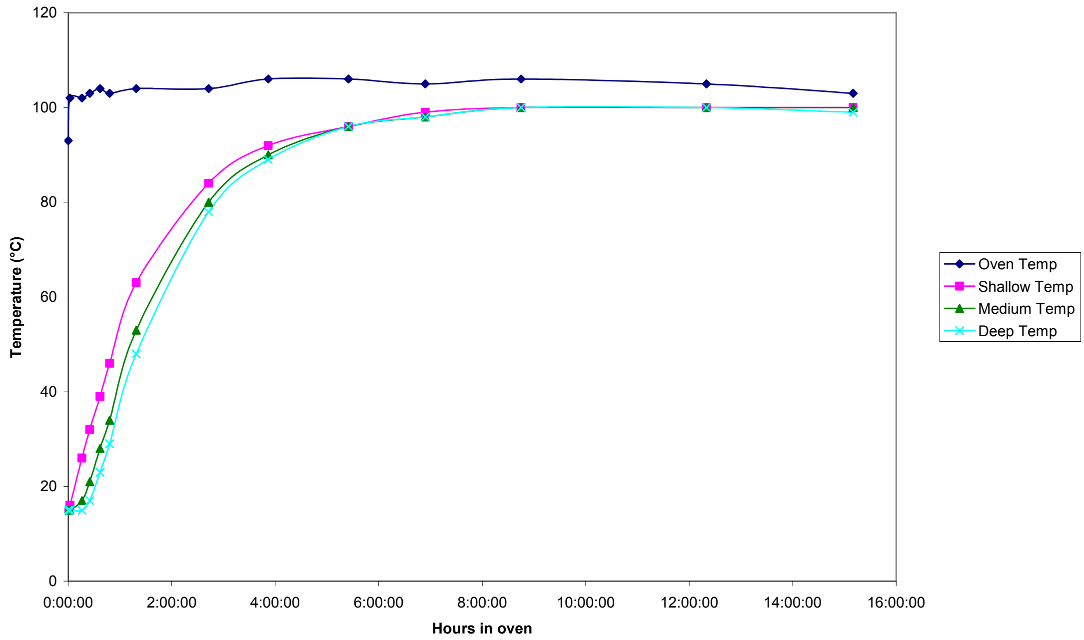


Figure 6.1 Specimen heating curve in oven

It can be seen from Figure 6.1 that after 16 hours in the oven the temperature within the epoxy had reached a steady state, approximately 3 degrees lower than that within the oven. Given that the thermostat on the oven allows the temperature within the oven to cycle over a range of around 5 degrees, this small variability was considered suitable.

It is also important to note that the temperature along the depth of the epoxy was constant and that the temperatures at all three/four measurement points had stabilised within 16 hours. These results showed that the proposed 16 hour heating of the specimens was sufficient to ensure that the temperature within the test specimen was the same as that of the oven.

6.1.2 Testing of specimens

To ensure that the test specimens did not cool prior to being tested they were individually unloaded from the oven immediately prior to strength testing. The testing frame was located only a few metres from the oven in order to reduce the time the test specimen was able to cool prior to being tested. A typical time for which a test specimen was out of the oven prior to testing was 2 minutes.

All the test specimens were tested in the custom-built testing frame, without the furnace attached, as shown in Figure 6.2. During these tests the load applied to the specimen and the corresponding extension were recorded electronically.

Figure 6.2 Custom-built testing frame, with oven test specimen loaded

Load was applied to the frame using a manual pump, which fed the hydraulic ram attached to the sample. As the load was applied manually the loading rate could not be accurately controlled, but attempts were made to be consistent between test specimens.

6.2 Results

All of the adhesives showed a decrease in ultimate strength with increasing temperature, as expected. The overall results from the oven testing are shown in Figure 6.3. It can be clearly seen that the HY 150 has the lowest ultimate strength of the three adhesives tested and that its strength reduces at a similar rate to two epoxies (West System and RE 500). For comparison purposes the results from the cold testing have also been included in Figure 6.3.

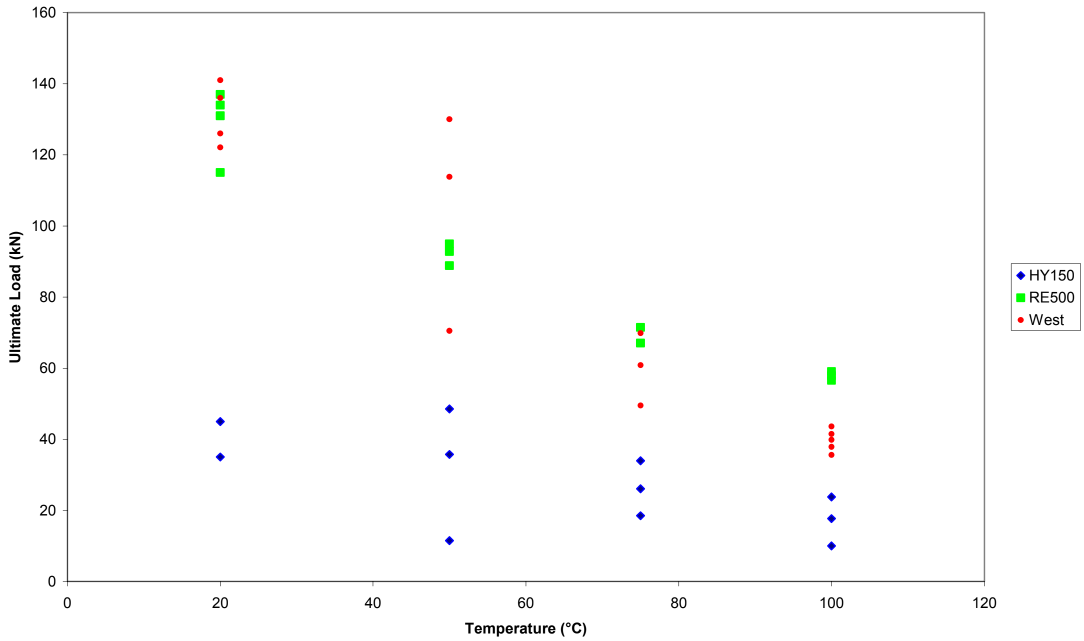


Figure 6.3 Oven test results, RE 500 results with air bubbles have been excluded

At 50 °C, the West System epoxy was found to hold its strength better than that for the RE 500 system. The 50 °C West System specimens gave results with ultimate loads only slightly lower than those at ambient temperatures. Ultimate loads for the RE 500 system initially decreased at a greater rate than the West System, but at 100 °C the RE 500 specimens have the greater ultimate strength of the two epoxies.

Some tests were performed on test specimens which had been glued with RE 500 without the use of the special nozzle described in 4.4.2.1. In many of these tests large air bubbles were found in the resin, which gave low strength specimens. The test specimens with air bubbles are identified in Table 6.3. It is not recommended that RE 500 samples are glued without the use of the special nozzle, due to the possibility of air bubbles in the resin. Because of this, the results for all of the RE 500 specimens with air bubbles have been excluded from Figure 6.3 and subsequent analysis.

To give a clearer indication of the decrease in strength of the adhesives at elevated temperatures the mean strength at each temperature for each of the adhesives has been plotted in Figure 6.4.

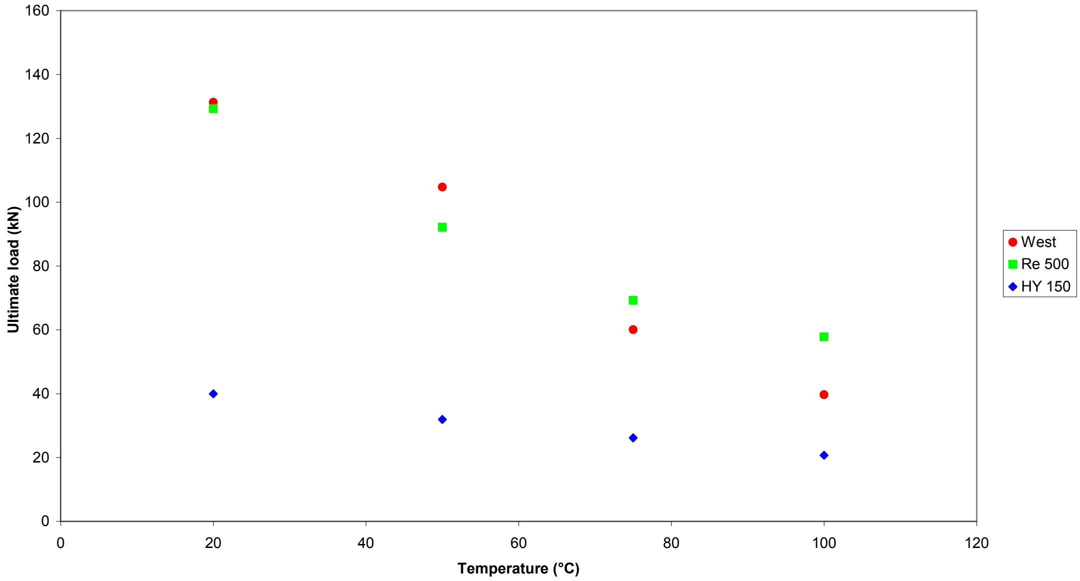


Figure 6.4 Mean oven test results, clearly showing strength decrease with temperature

It was observed in the results that the HY 150 specimens contained a lot of scatter, with considerably more spread than for the RE 500 and West System epoxies. Investigation of the failed HY 150 specimens found no evidence of air bubbles in the adhesive, nor any other factors which might have contributed to a reduction in strength. It is concluded that the wide variation in the ultimate strength between HY 150 replications was primarily due to the poor bond between the adhesive and timber. This poor bond is thought to be affected by possible shrinkage and swelling associated with the oven heating regime.

6.2.1 Failure modes

Three modes of failure were observed in the specimens tested in this phase of the research. As two of the failure modes were consistent with those observed in the cold testing component of this research the labelling of these failure modes will continue from those described in section 5.3

Failure modes for each of the three adhesives are tabulated in Table 6.2 through Table 6.4. The failure modes observed in oven testing are described in the following sections.

Table 6.2 Failure modes and ultimate loads from oven testing of West System

Sample	Adhesive	Test temperature (°C)	Screws	Failure Load (kN)	Failure mode	Comments
23	West System	50	In wood	114	3	
24	West System	50	In wood	130	3	
65	West System	50	In wood	70	3	
25	West System	75	In wood	70	4	
26	West System	75	In wood	61	4	
71	West System	75	In wood	50	4	
27	West System	100	-	36	5	
28	West System	100	-	44	5	
73	West System	100	In wood	41	4	
74	West System	100	In wood	38	4	
75	West System	100	In wood	40	4	

Table 6.3 Failure modes and ultimate loads from oven testing of RE 500

Sample	Adhesive	Test temperature (°C)	Screws	Failure Load (kN)	Failure mode	Comments
29	RE 500	50	In wood	95	3	
30	RE 500	50	In wood	89	3	
42	RE 500	50	In wood	67	3	Air bubble
44	RE 500	50	In wood	93	3	
33	RE 500	75	In wood	71	4	
34	RE 500	75	In wood	56	4	Air bubble
55	RE 500	75	In wood	55	4	Air bubble
59	RE 500	75	In wood	67	4	
31	RE 500	100	-	27	5	Air bubble
32	RE 500	100	-	59	5	
63	RE 500	100	In wood	57	4	
64	RE 500	100	In wood	58	4	

Table 6.4 Failure modes and ultimate loads from oven testing of HY 150

Sample	Adhesive	Test temperature (°C)	Screws	Failure Load (kN)	Failure mode	Comments
37	HY 150	50	-	49	2	
38	HY 150	50	-	36	2	
50	HY 150	50	-	11	2	
35	HY 150	75	-	34	2	
36	HY 150	75	-	26	2	
47	HY 150	75	-	19	2	
39	HY 150	100	-	17	2	
40	HY 150	100	-	24	2	
48	HY 150	100	-	10	2	

6.2.1.1 Mode 1 failures

Mode 1 failures were not observed in this phase of the research.

6.2.1.2 Mode 2 failures

Mode 2 failures were observed for all of the HY 150 specimens tested in the oven. This indicates that the temperatures reached during this phase of testing, although high enough to cause a loss of strength, were not great enough to alter the behaviour of the HY 150 adhesive.

6.2.1.3 Mode 3 failures

Mode 3 failures were recorded in the epoxy specimens (West System and RE 500) at ambient temperatures when the screws were included to prevent splitting of the laminations. This failure mode was also observed at 50 °C for both brands of epoxy resin. This indicates that at 50 °C the epoxy resins, although they had lost strength, were still behaving as they did at ambient temperatures.

6.2.1.4 Mode 4 failures

Mode 4 failures occurred in the epoxy resin specimens once the adhesive had undergone some thermal degradation. In this mode the failure occurred within the resin, with no breaking of timber. Associated with this mode of failure, the epoxy was very crumbly and could easily be brushed from the surface of the withdrawn rod with the fingers.

This mode of failure was similar in the two epoxy resins and can be seen in Figure 6.5, with a RE 500 specimen on the left and a West System specimen on the right. Both of these specimens were tested at 100 °C, identical to those tested at 75 °C.

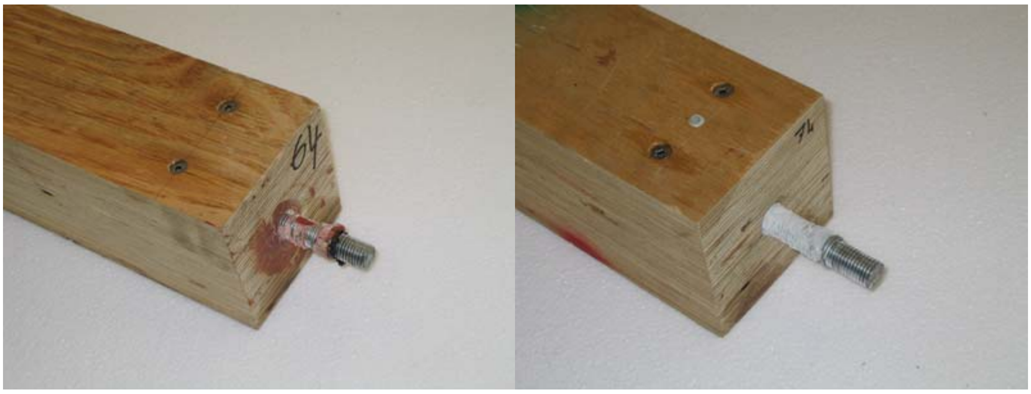


Figure 6.5 Mode 4 failures in RE 500 specimen (left) and West System specimen (right). These specimens were tested at 100 °C

The transition to the epoxy being crumbly was most likely due to the resins being heated beyond their heat distortion temperatures (HDT). For each brand of epoxy there is a HDT, which is a measure of the epoxy's reduction in stiffness when heated (Goulding, 1994). For the West System this is 60 °C (Adhesives Technologies, 1992) and for the RE 500 system the HDT is not published, but it has a maximum service temperature of 50 °C (D. Arandjelovic, pers. comm.). Once this temperature is exceeded the crystal structure of the epoxy begins to deform and it is thought that this deformation combined with the removal of the steel bar caused the crumbling of the epoxy that was observed.

6.2.1.5 Mode 5 failure

Mode 5 failures were a special case of mode 4 failures. In construction of the test specimens which were tested at 100 °C it was decided that screws would not be required to prevent splitting between the laminations. In making this decision a predicted failure load based on the results of the 50 and 75 °C tests was calculated. This failure load was less than half of that which the mode 1 failures to occur during the cold tests. Based on this theory the screws between the laminations to prevent splitting should not be required. This proved to be an incorrect hypothesis, as the epoxy specimens without screws tested at 100 °C were observed to split along a plane parallel to the laminations. This mode of failure is shown in Figure 6.6 for a West System specimen.

Figure 6.6 Mode 5 failure in a West System specimen

Although the split shown in Figure 6.6 did not open particularly wide it is worth considering as it demonstrates a loss in strength within the timber. As the split lies on the same plane as the glue between the laminations in the timber it is necessary to identify whether the failure was a failure of the glue, or of one of the laminates.

Quality assurance testing of LVL following production by Carter Holt Harvey involves a test to check bond strength. These tests include boiling the LVL and then chiselling laminates from the test specimen. During these tests failures occur at the weakest point, either within the glue line and the timber laminate. To identify which of the two materials was the location of the failure visual identification was used. This visual identification was based on the proportion of the resorcinol adhesive exposed compared to the amount of timber exposed. To pass this test the specimen needed to have a maximum of 50% of the exposed surface (failure surface) being resorcinol (H. Bier, pers. comm.).

In making this comparison with the failures in this research one of the specimens with a mode 5 failure was cut to allow the failure surface to be exposed. This exposed surface is shown in Figure 6.7.

Figure 6.7 Exposed failure surface for mode 5 failure

When compared to the guidance for the quality assurance testing, this surface was rated as having 60 – 70 % failure within the timber. This means that there was no more glue failure than would be expected of a section of LVL leaving the factory. This suggests therefore that it is the timber which is losing strength.

Investigations by Green *et al.* (1999) have identified the loss in timber strength with increasing temperature. They noted that at 50 °C the tensile strength perpendicular to the grain of clear wood (unspecified species, 11 – 16% moisture content) can be 20 % less than at 20 °C. Their results did not include a reduction in strength for 100 °C, but it is conceivable that the magnitude of strength loss observed in these tests could be due solely to the temperature of the timber.

To prevent the mode 5 failures from occurring the screws perpendicular to the laminates do need to be included in the test specimens. Further testing at 100 °C with screws perpendicular to the laminations (tests 63, 64, 73, 74 and 75, refer Table 6.2 and Table 6.3). These tests all gave mode 4 failures, which indicated that the screws worked effectively. However, the ultimate loads were not observed to be significantly greater than those specimens tested without the screws at 100 °C.

As both the mode 4 and mode 5 failures involve epoxy failure it may be that the crumbling of the glue is the primary limit on the strength of the test specimen and it just happened to coincide with the reduced strength of the laminations. Even though at 100 °C it was observed that there is no difference in strength whether there are screws between the laminations or not, it is recommended that these screws are used as they make a significant improvement to the strength under normal conditions.

6.3 Cooled tests

Consultants working in the field of fire engineering have unanswered doubts about the use of epoxy-grouted bars in timber construction, as there is considerable uncertainty as to whether the connection will regain its strength following a fire. In a moderate fire heavy timber members will often survive with only a small degree of charring. However, if the connections to these members do not regain their strength following a fire then the connections and possibly the timber members would have to be replaced, with the building owners incurring significant cost and downtime.

Barber (1994) briefly examined this problem by subjecting failed glulam specimens, with epoxy-grouted rods, to a cold test. These specimens had been loaded in tension while being exposed to the standard fire in the furnace at the Building Research Association of New Zealand (BRANZ). During the furnace test the specimens all failed and displaced by around 20mm. These test specimens were then allowed to cool prior to being subjected to a tensile test at ambient temperature. Barber (1994) noted that the residual strength of these specimens was between 2 and 22 kN. This was a reduction from a cold strength of approximately 120 kN. Barber (1994) concluded that “once the epoxy connections have been severely heat affected that their strength properties are greatly reduced”.

It could be argued, however, that Barber’s (1994) testing was not a fair test of the residual strength of the connection following heating, as the test specimen had failed prior to the cooled test being performed. In order to address this weakness in Barber’s methodology test specimens were tested in this research which had been built specifically to test the heating and cooling process.

Residual loads recorded by Barber (1994) for specimens which had been heated and cooled were recorded to be around 20 kN. During the cold testing phase of this research test specimens were often loaded beyond their failure point in order to withdraw the rods further for inspection. Loads recorded during this process were around 10 – 20 kN. This indicates that the loads found by Barber as residual loads following heating are actually likely to be residual loads following failure of the specimen. The force being recorded is not so much the residual strength of the connection, as the dynamic friction force caused by the rod being withdrawn from the specimen.

By this comparison it can be seen that it is not so much the heat that is creating the loss in strength, but the displacement of the test specimen.

Similar research was carried out by Stumes (1982) in which steel pins were grouted into timber specimens and heated in an oven. Stumes' testing was different from both that done in this study and that done by Barber (1994). Stumes heated the test specimens in an oven with a constant load applied and did not hold them there until failure. Stumes does not specify the length of time the specimens were left in the oven, nor the load applied to them. He concludes that if epoxy is heated and cooled then its strength is permanently reduced (Stumes, 1982). As epoxy will creep when it is loaded at higher temperatures, as seen in the furnace testing section of the current research, it may be that the displacement caused by the loads while the specimens were in the oven was sufficient to displace the test specimen enough that the epoxy structure was damaged upon cooling.

The current study therefore set out to test the theory that it was displacement which was critical to the weakening of the epoxy, not the temperature. To do this the current study involved the heating of test specimens to 75 and 100 °C in the oven. These test specimens were then left overnight at this temperature to ensure that they achieved a constant temperature, were then removed from the oven and allowed to cool to room temperature. Once the test specimens had reached room temperature they were then subjected to a tensile test.

Results of these tests can be found in Figure 6.8 to Figure 6.10 and show the comparison between the test specimens tested hot and those which had been allowed to cool fully prior to testing. It can clearly be seen that for each of the three adhesives there is no significant difference in strength between those test specimens which have been tested at ambient temperature and those which were heated and then tested at ambient temperature. This is important as it shows that the epoxy can withstand the heating and cooling process that would likely be experienced during a minor fire.

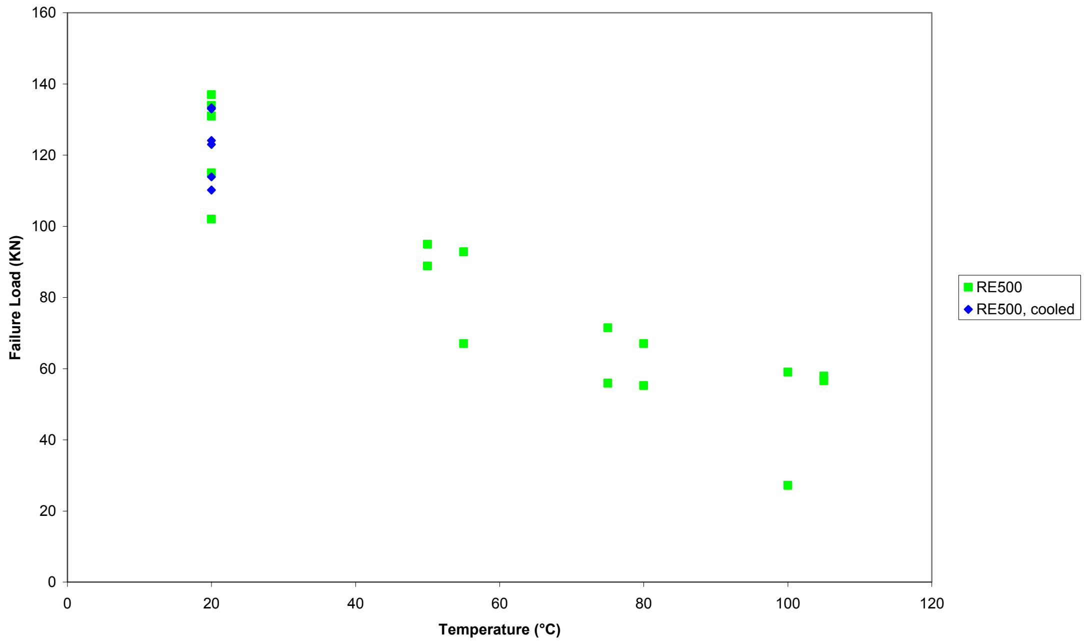


Figure 6.8Comparison of RE 500 failure loads at elevated and cooled temperatures

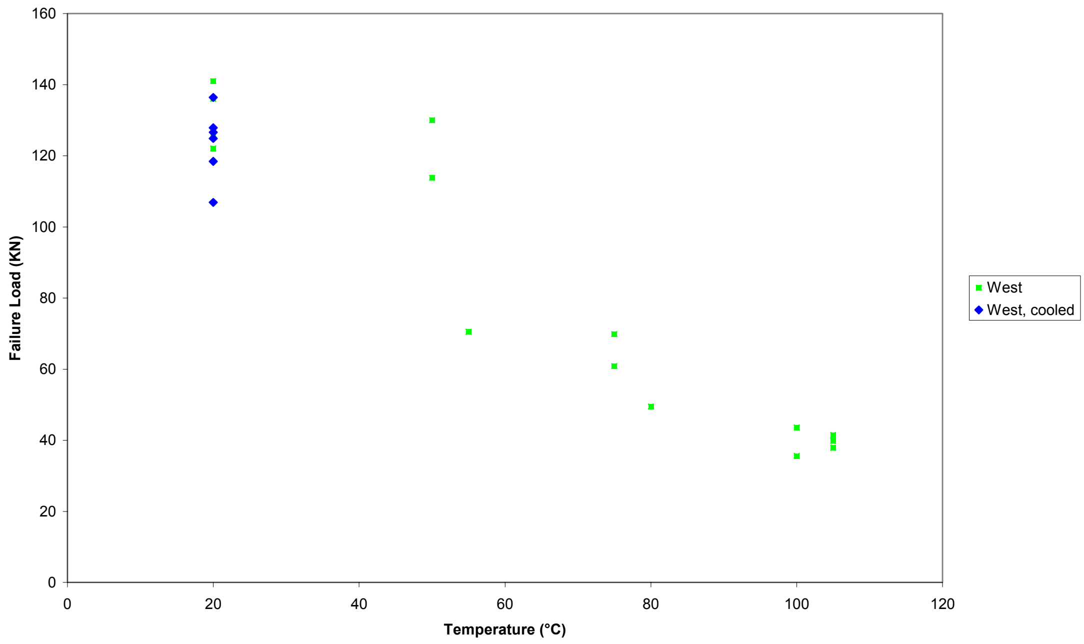


Figure 6.9 Comparison of West System failure loads at eleveated and cooled temperatures

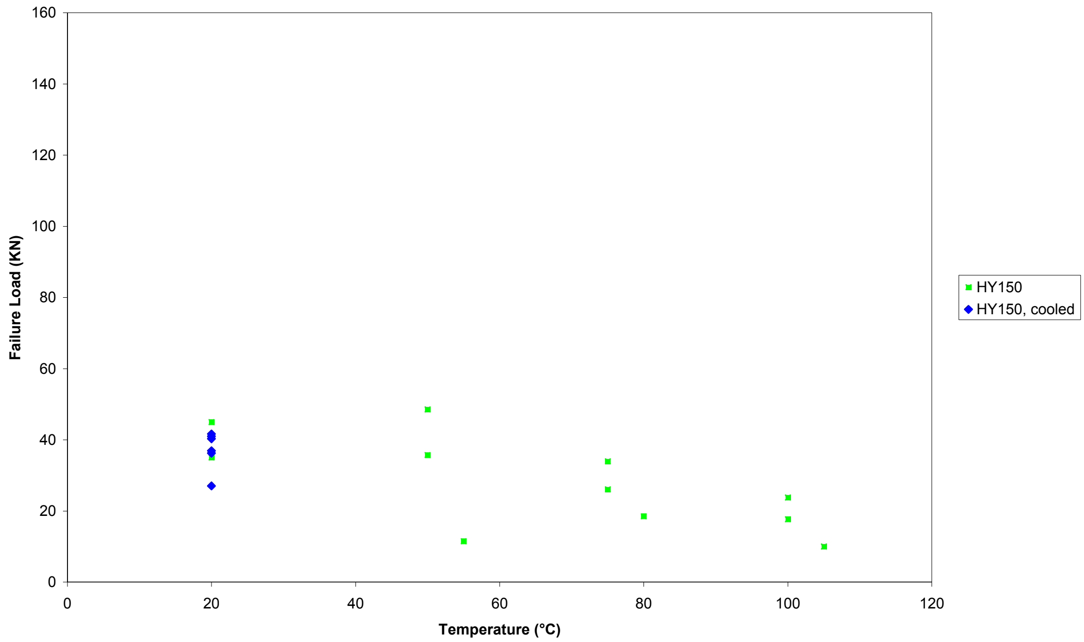


Figure 6.10 Comparison of HY 150 failure loads at eleveated and cooled temperatures

Within the current study there were no differences found between test specimens that had been heated to 75 and those that had been heated to 100 °C. The current study also heated the epoxy to a greater temperature and maintained it at that temperature for a longer period of time than Barber (1994) had used in his testing. The epoxy in Barber's (1994) test specimens reached 40 – 45 °C and the temperature was only maintained for around 30 minutes. This means that the heating performed in this study was more severe than that used by Barber (1994) and the specimens performed better in this study, which means that the prior failures of Barber's specimens was critical to their low residual strength.

6.3.1 Failure modes

When tested at 75 and 100 °C the failure of the RE 500 and West System epoxy resin specimens was found to be within the epoxy. At these temperatures the adhesive became visibly crumbly. This is in contrast to the failure modes at ambient temperatures where the epoxy specimens failed within the timber. Resin failure (mode 4) was visible on the failed oven-heated test specimen, where the adhesive had crumbled and was able to be brushed away with the fingers.

On the test specimen which had been heated and cooled there was no evidence of the epoxy being crumbly, as was seen in the hot tests. Failure of all of the heated and cooled epoxy specimens were mode 3 failures. For the hot test specimens where crumbling of the epoxy was observed it was noticed that after cooling the texture of the epoxy remained crumbly.

It is concluded that the crumbling of the epoxy is not solely a function of temperature and that it must be as a result of load being applied while the epoxy was at an elevated temperature. This conclusion is also supported by the condition of the overflowed epoxy on the surface of the test specimens surrounding filling holes from the gluing process. This epoxy was exposed to the same temperatures as that in the connection and it never showed any sign of being crumbly. Efforts to remove it by hand were futile indicating that it had regained its strength upon cooling, unlike the crumbled epoxy seen on the withdrawn rod.

Heated and cooled tests using HY 150 gave similar results to the epoxy specimens (West System and RE 500) in that the behaviour was observed to be the same as for tests at ambient temperatures. Failures in the HY 150 were mode 2 failures at ambient and elevated temperatures and mode 2 failures were again observed in the heated and cooled tests.

6.4 Conclusions

From this phase of tests it can be seen that there were some differences between the performance of the two epoxy resins (RE 500 and West System) at elevated temperatures. Strength losses for the two epoxies were similar. Both were reduced to around 40 % of their cold strengths at 100 °C. The West System maintained its strength better at 50 °C, but beyond this point the RE 500 was the stronger of the two. The difference was not great enough for the RE 500 to be recommended in favour of the West System when temperature resistance is required.

There was an observed difference in consistency in the epoxies when they were heated to 75 °C and above. At this temperature the epoxy made a transition from being solid to being crumbly. It is thought that this transition was due to the epoxies having exceeded their heat distortion temperatures.

This occurrence was not observed for the HY 150 tests where the failure modes remained at the wood/glue interface as they had been at ambient temperatures. Failure loads in the HY 150 decreased with increasing temperature, but at a lesser rate than for the epoxies.

Overall, the HY 150 lost strength at a lower rate than the two epoxy resins, but had highly variable strength, attributed of the lack of bonding.

7 Furnace testing

The objective of this phase of testing was to measure the fire resistance of the epoxy-grouted steel rod connection in LVL timber. This was done by testing the specimens at a constant tensile load while being heated until failure in a furnace. This contrasted with the oven testing phase where the temperature was held constant and the load was increased until failure.

7.1 Construction of specimens

As the insulation provided to the adhesive by the timber is important in this testing there were two specimen sizes tested. One group of test specimens were 105 x 105 mm and were constructed to the same specifications as for the oven test specimens. The other group were smaller, 63 x 63 mm test specimens. All test specimens were the same length (900 mm) and the rods were grouted to a depth of 300 mm with 20 mm holes.

The size of the smaller specimens was chosen as it was a more realistic size of timber for the capacity of the connection. For a 63 x 63 mm section of LVL, the tensile strength, based on a yield strength of 42 MPa (CHH 2000) is 167 kN. This was similar to the ultimate load for the 16 mm epoxy-grouted rod of 130 kN.

For data recording purposes a type K thermocouple was cast into the adhesive in all of the test specimens. This thermocouple gave temperature readings of the adhesive at the midpoint of the hole (embedment depth of 150 mm) and was connected to the electronic data logging equipment to give readings every second throughout testing.

7.2 Testing frame

The same testing frame used for the oven tests was used for these tests, but now a furnace was added, as shown in Figure 7.1. Load was applied using the same mechanism as for the oven tests, except that in the furnace testing the load was held constant throughout.

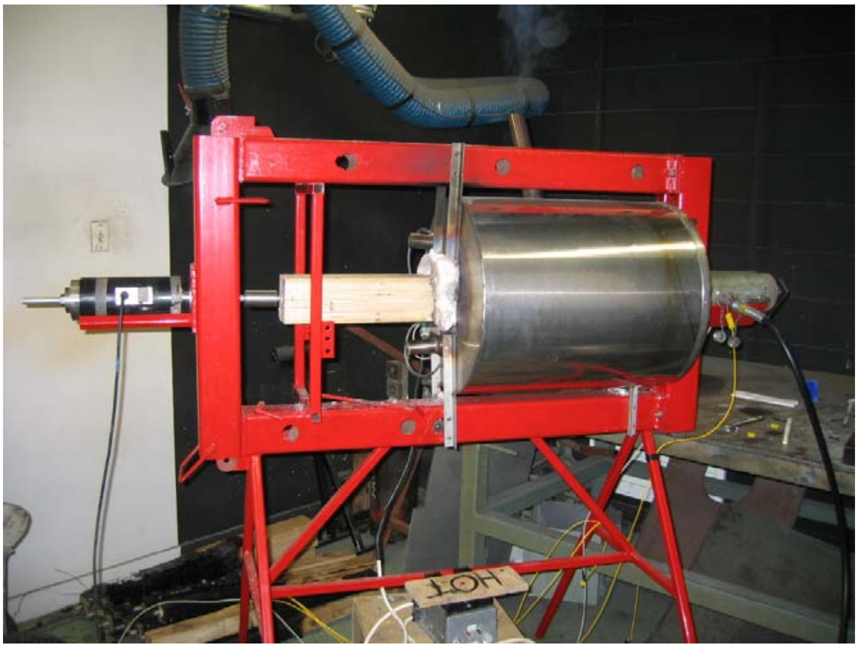


Figure 7.1 Custom-built testing frame with furnace attached

To ensure that the specimens were exposed to the same heating conditions in every experiment the oven was pre-heated to 300 °C and held at that temperature for 10 minutes prior to the test specimen being placed into the furnace. This temperature was recorded by inserting a thermocouple into the centre of the oven. By holding the furnace at 300 °C for 10 minutes this allowed the body of the furnace to heat up. During trials of the furnace it was found that the first specimen tested in a series was subjected to a less severe test unless the furnace was allowed to heat thoroughly.

Following a test the furnace was often at temperatures in excess of 300 °C, so it was necessary to allow it to cool down to the 300 °C starting point, again to ensure a constant starting point for all of the tests. Verification of the similarity of heating regimes between each of the tests can be found by plotting temperature time plots for the temperature within the test specimens during testing, as shown in Figure 7.2.

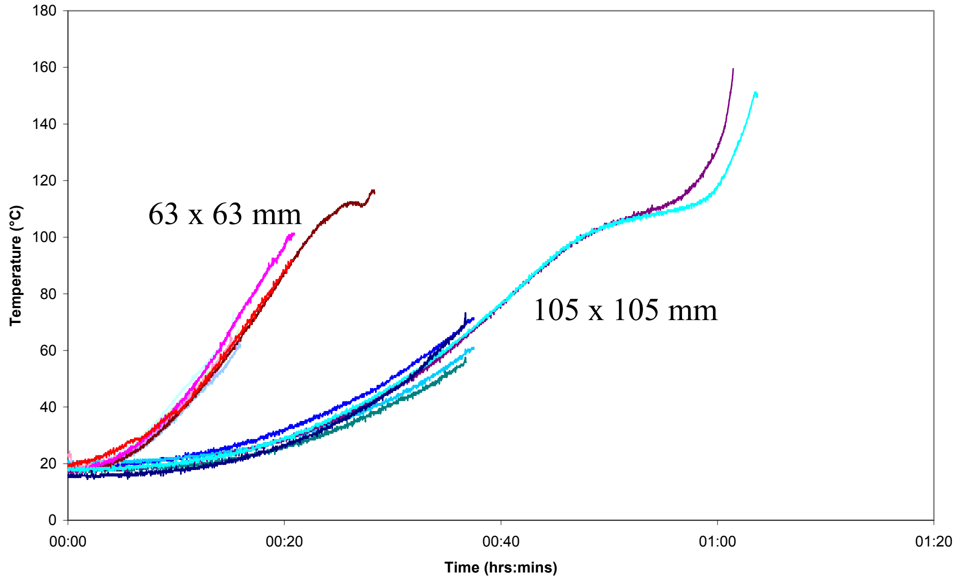


Figure 7.2 Temperature - time plot showing heating of test specimens in furnace.

It can be seen in Figure 7.2 that although the end points of each of the curves differ the curves are closely grouped indicating very similar heating regimes. The difference in end point is due to variations in applied load between different adhesives, as explained in section 4.5.3.1.

7.2.1 Insulation of test specimen

It was anticipated that the heat would travel solely through the thickness of the timber into the epoxy. For this reason it was important that element was not able to directly heat the rod. If the rod had been heated then it could be expected that with the higher thermal conductivity of the steel, heating of the epoxy would be mostly due to heat transfer along the rod rather than through the timber.

Insulation of the rods at the heated end was achieved by inserting a piece of 40 mm thick Kaowool mineral insulation over the exposed steel rod, as shown in Figure 7.3.

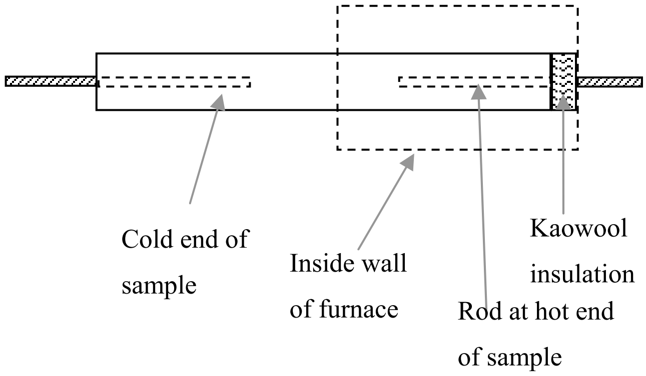


Figure 7.3 Schematic showing location of Kaowool insulation

Following tests it was seen that this insulation was effective as no charring was visible at the end surface of the test specimens. This is evidenced in Figure 7.4

Figure 7.4 Photo showing uncharred end of burnt specimen where protected by Kaowool insulation. The exposed screws show the extent of timber burnt away

To keep the heat within the furnace an additional seal of Kaowool insulation was provided around the test specimen at the opening of the furnace. This insulation provided a heat and gas seal around the sides and top of the test specimens, but left the bottom surface open. It was found during trials of the furnace that some ventilation was required to feed the combustion. Without this air supply the ignition of the test specimen was dangerously explosive.

The test specimen was positioned in the furnace and the air supply was regulated such that the heated specimen had approximately uniform charring n all four surfaces.

7.3 Charring tests

A series of charring tests were performed to investigate the consistency of the heating in the furnace. These tests consisted of loading the furnace with 105 x 105 mm specimens (previously failed during oven tests), but without a tensile load applied. These specimens were then heated in the furnace for 15, 30 or 45 minutes, with two replications at each duration. Following these tests the specimens were removed from the oven, extinguished and cooled.

For observation purposes these test specimens were then cut open (as shown in Figure 7.5). The layer of char was then removed to expose the remaining section for measurement, as seen in Table 7.1.

Figure 7.5 Charring specimens after being cut open. From the left there are two specimens exposed for 15 minutes, followed by two at 30 minutes and two at 45 minutes. Note that the top layer of specimens are the matching halves to the lower layer

From Figure 7.5 it can be seen that the two 45 minute specimens have similar residual sections, but that at 15 and 30 minutes there were differences between the replicates. For the 15 minute test specimen (second from left in Figure 7.5) there was insignificant charring to the lower surface. This was found to be due to incoming air cooling the specimen and was remedied as described in section 7.2.1. It can also be seen that one of the 30 minute specimens had less charring. This test specimen was the first test specimen tested one morning and although the furnace had been pre-heated to 300 °C prior to loading, it had not been hot long enough to sufficiently heat the body of the furnace. This was remedied by allowing the furnace to heat through prior to use, as described in section 7.2.

Table 7.1 Char depths for charring specimens

Sample	Exposure (minutes)	Time to charring (minutes)	Time to ignition (minutes)	Depth of char (mm)	Average char rate (mm/min)
50	15	6:30	Did not ignite	4	0.27
52	15	2:30	10:00	9	0.6
51	30	10:00	21:00	14	0.47
54	30	3:00	12:00	19	0.63
49	45	4:00	18:00	25	0.56
53	45	3:00	15:30	25	0.56
			Overall average		0.52
			Overall average without 50 & 51		0.59

If specimens 50 and 51 are discounted due to their being tested under less severe conditions, it can be seen that the average charring rate was 0.59 mm/min. This was slightly lower than that achieved by Lane *et al.* (2004) in a pilot test furnace heated in accordance with ISO 834, where an average charring rate of 0.7 mm/min was achieved for similar LVL timber.

The measured charring rates gave an indication that the furnace was less severe than the ISO 834 heating regime. A scaling factor was used to convert the results of the furnace testing to those achieved in a standard furnace test based on the depth of charring achieved.

7.4 Results

During testing of the specimens a slow decrease in load was observed with increasing temperature. Some of this was attributed to a slight leak in the hydraulic pump, but mostly it was found to be due to extension of the test specimen, as recorded by the potentiometer. A typical load vs. displacement graph is shown in Figure 7.6 for an epoxy test specimen tested in the furnace, which shows the load fluctuations around the 27 kN target.

Constraints due to the design of the furnace prevented recording of the displacement between the rod and the test specimen and so from the results it was not possible to differentiate extension of the timber from slip of the adhesive. Analysis of this could only be made after completion of the test through inspection of the position of the rod relative to its original position.

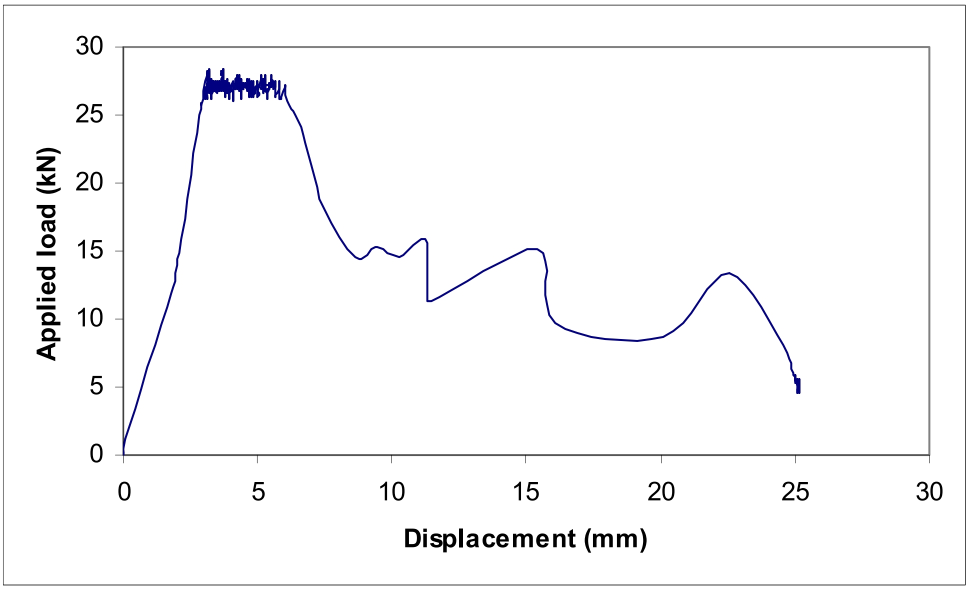


Figure 7.6 Typical load vs. displacement graph for furnace test

Unfortunately, due to a serious malfunction of the oven the furnace testing for this phase of the research was unable to be completed. Due to this failure only 15 of the intended 18 tests were completed.

7.4.1 Burning behaviour

Following insertion into the furnace it took all specimens between 2 and 5 minutes to heat sufficiently to begin charring. As the surfaces of the test specimens were not visible from the outside of the furnace, the onset of charring was only evidenced as smoke began to be expelled from the flue. This led to a period where the test specimens were charring and building up pyrolyzates within the furnace, but prior to flaming occurring. After the pyrolyzates built up for a few more minutes the gases reached their unpiloted ignition temperature and ignited. From the point of ignition, there was flaming on all surfaces of the test specimen until the conclusion of the test.

As soon as the test specimens had failed the furnace was switched off and the test specimens were removed from the furnace. Once extracted from the furnace the test specimens were quickly extinguished and cooled with water to prevent further charring.

7.4.2 Time to failure

Zero time was recorded as the time at which the 27 kN or 8 kN test load was applied. Failures were recorded as being the time at which the test load was not able to sustained by the test specimen.

Recorded times to failure for each of the tested configurations are shown in Table 7.2 with their associated modes of failure.

Table 7.2 Time to failure for furnace testing

Sample	Sample size	Adhesive	Test load (kN)	Time to charring (minutes)	Time to ignition (minutes)	Time to failure (minutes)	Failure mode
79	63 x 63 mm	West	27	2:30	14:00	15:00	6
80	63 x 63 mm	West	27	4:30	14:00	15:30	6
81	63 x 63 mm	West	27	3:00	12:00	15:30	6
82	63 x 63 mm	RE 500	27	4:00	13:00	16:00	6
83	63 x 63 mm	RE 500	27	4:30	12:00	15:00	7
84	63 x 63 mm	RE 500	27	4:00	11:30	15:30	6
85	63 x 63 mm	HY 150	8	2:00	12:00	28:30	6
86	63 x 63 mm	HY 150	8	3:30	11:30	20:30	4
87	63 x 63 mm	HY 150	8	3:30	12:30	21:00	4
76	105 x 105 mm	West	27	5:00	18:00	38:00	4
77	105 x 105 mm	West	27	2:00	16:00	36:30	4
78	105 x 105 mm	West	27	3:00	17:30	38:00	4
91	105 x 105 mm	RE 500	27	2:00	13:00	36:30	4
88	105 x 105 mm	HY 150	8	2:30	16:00	61:30	4
89	105 x 105 mm	HY 150	8	3:00	17:00	63:30	4

7.5 Failure modes

The failure modes observed in the furnace testing were slightly different from those observed in the earlier phases of this research. Again, for consistency the numbering of the failure modes continues from the earlier phases of this research.

Failure modes 1 and 3 were not observed in this phase of testing as the applied load was not enough to cause timber failure. Mode 2 failures were also not observed. This was slightly unexpected, as the HY 150 specimens preceding the furnace tests had all failed in the same manner, at the wood/glue interface.

7.5.1 Mode 4 failures

Mode 4 failures occurred, as described in the oven testing results, with a failure of the epoxy resin and no damage to the timber. This mode of failure was evidenced in all of the 105 x 105 mm tests and in two of the three 63 x 63 mm tests with HY 150 adhesive only.

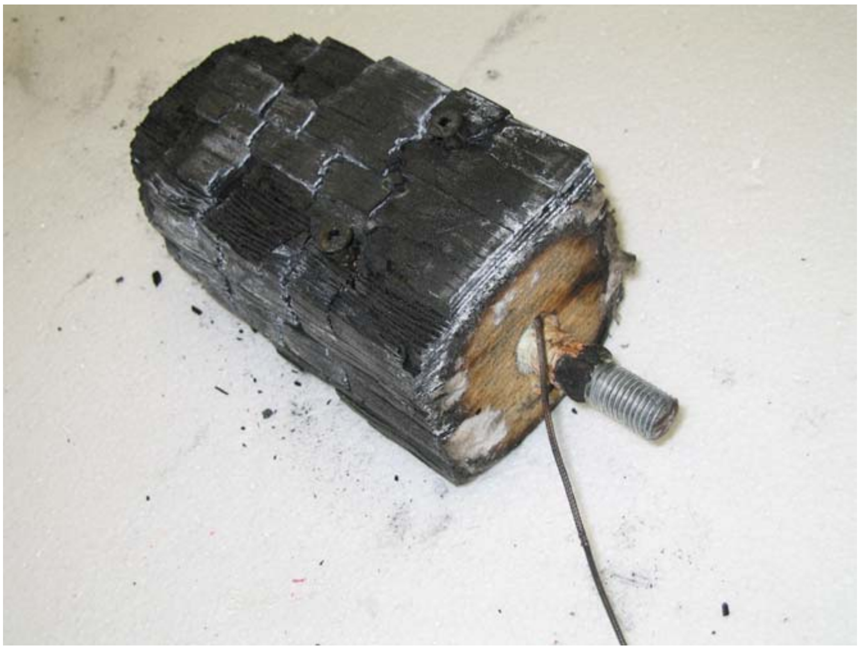


Figure 7.7 Mode 4 failure in portion of a West System 105 x 105 mm specimen

This indicates that the insulative properties of the timber were sufficient to protect the epoxy until the time of failure.

7.5.2 Mode 6 failures

Mode 6 failures involved the failure of the adhesive, but the splitting of the test specimen only occurred on one side. Mode 6 failures are very similar to the mode 5 failures observed in the oven testing phase. Mode 5 failures occurred within the adhesive combined with a splitting across the laminations, on both sides of the steel rod.

One important point to note about mode 5 and 6 failures is that mode 5 failures occurred in specimens without screws perpendicular to the laminations to prevent splitting. In the test specimens where mode 6 failures occurred there were screws perpendicular to the laminations, although due to the mode of failure there was doubt as to their effectiveness.

A West System test specimen demonstrating a mode 6 failure can be seen in Figure 7.8. The crack can be observed in the horizontal plane on the left of the rod.

Figure 7.8 Portion of 63 x 63 mm West System specimen showing mode 6 failure

In the HY 150 specimens the specimens charred deeper (but at a similar rate) than their epoxy (West System and RE 500) counterparts, due to their longer exposure to the furnace. In many specimens this led to there being very little, if any, residual section remaining following the testing. Specimen 85 was the only one of the three 63 x 63 mm HY 150 specimens to have a mode 6 failure.

Figure 7.9 Extreme case of mode 6 failure, in a 63 x 63 mm HY 150 specimen

7.5.3 Mode 7 failures

Mode 7 failures were when the test specimens split perpendicular to the laminations. This was only found to happen in some of the furnace test specimens. Mode 7 failures were similar to mode 6 failures, except that they demonstrated greater effectiveness of the screws at preventing splitting between the laminates. As the splitting force was not able to part the test specimen along the line of the laminations it split across the laminations instead. This can be seen in Figure 7.10, with the split occurring on the top surface.

Figure 7.10 Portion of 63 x 63 mm RE 500 specimen with split across laminations in top surface, demonstrating a mode 7 failure

7.6 Comparison with oven testing

Failure temperatures within the adhesive observed during the furnace testing were significantly lower than those observed during the oven testing. This was observed for all three adhesives and the comparison of these temperatures for each adhesive can be seen in Figure 7.11 through Figure 7.13.

For the epoxy resins (West System and RE 500) the temperature of the glue at failure was approximately half of that predicted by the results of the oven testing.

For the HY 150 specimens the difference between the oven and furnace results was not as great. There was found to be a large amount of scatter in the failure temperatures for the HY 150 (seen in Figure 7.13), which makes comparisons with the oven test results difficult. It was expected that this scatter was due to the poor bond of the adhesive to the timber being affected by moisture movement in the timber.

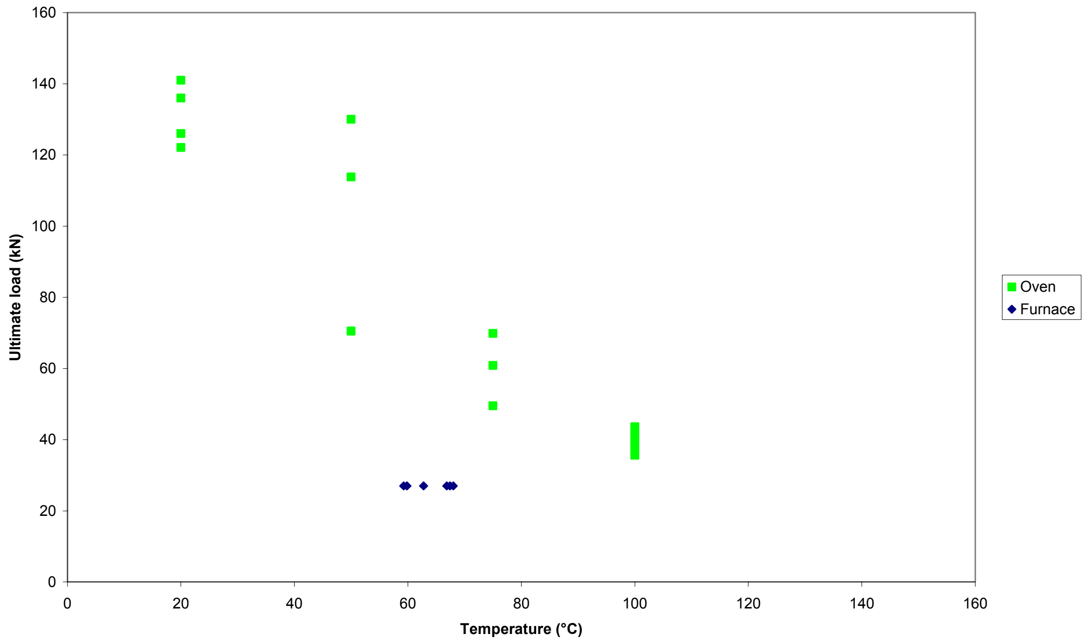


Figure 7.11 Comparison of oven and furnace ultimate loads for West System epoxy

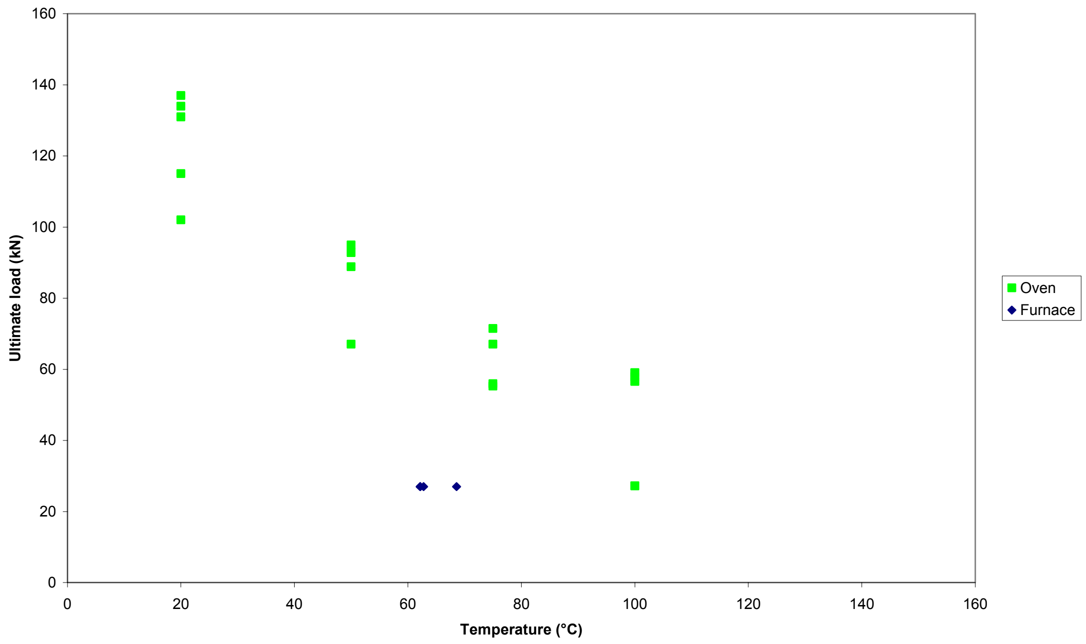


Figure 7.12 Comparison of oven and furnace ultimate loads for RE 500

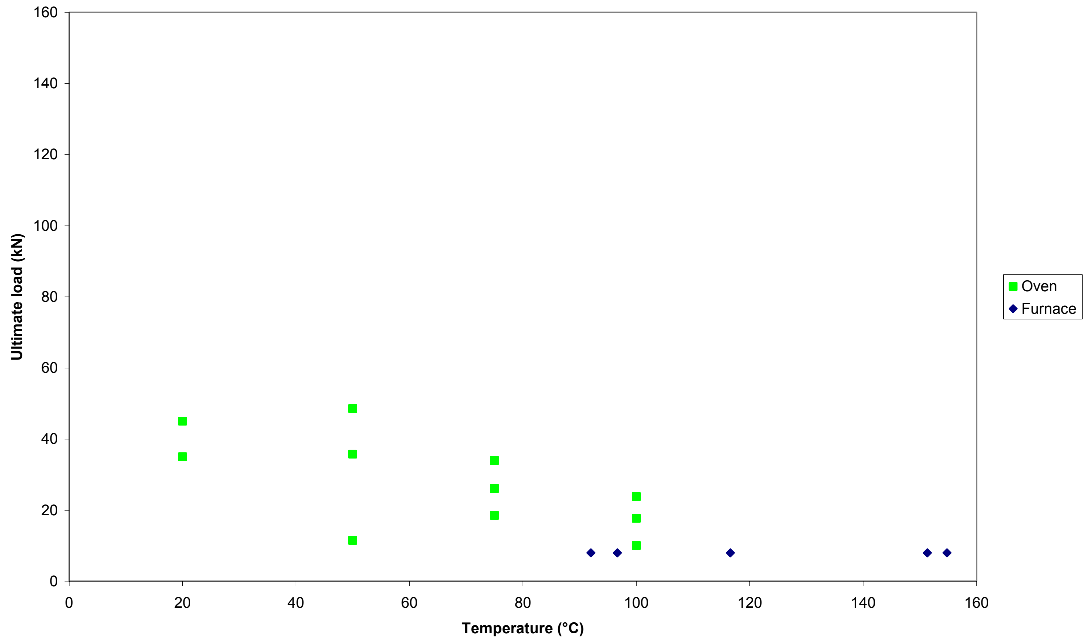


Figure 7.13 Comparison of oven and furnace ultimate loads for HY 150

The furnace testing was observed to be a more severe test for the specimens, when epoxy temperatures are compared between tests. This may be due to the moisture movement within the test specimen in the furnace. When specimens are heated beyond 100 °C the internal moisture is driven off. Initially some of this moisture will be forced towards the core of the timber. In the furnace tests this will increase the moisture content in the area of the bond. For the oven tests, as the test specimens were left in the oven for a longer period of time the moisture would have been able to reach an equilibrium point and so there would not have been the higher concentrations of moisture at the bond area.

Possibly more important is that during the furnace tests the adhesive was under tensile load while the heating was conducted. This would have placed stress on the structure of the adhesive as it was beginning to soften, as opposed to the oven test specimens where the adhesive was unloaded while the heating was occurring and load was applied after heating was finished.

Although the reason for the difference between the oven and furnace tests is not entirely clear, it was also observed by Barber (1994) and Spieth (2002) that furnace tests were more severe than oven tests.

7.6.1 Displacements during testing

An important factor of this testing was not just the load and temperature at failure, but also whether the displacements prior to failure were within an unacceptable range. While a definitive assessment of acceptable displacement was difficult to make when assessing only a tensile specimen in isolation from the complete structural assembly, it was possible to make a qualitative assessment.

Displacements were recorded during testing and, when compared to the temperature within the adhesive, gave an indication of the amount of slipping that was occurring. The displacements for each of the three adhesives can be found in Figure 7.14 through Figure 7.16. Displacements are missing for two tests, as there was an error in the data logging during the experiment.

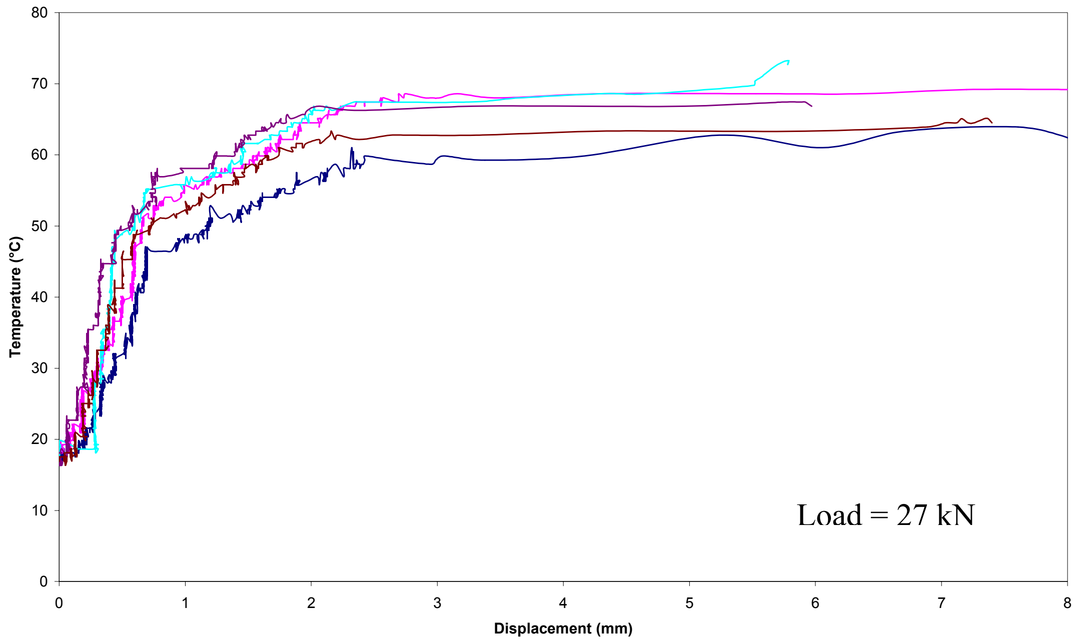


Figure 7.14 West System displacements recorded during furnace testing

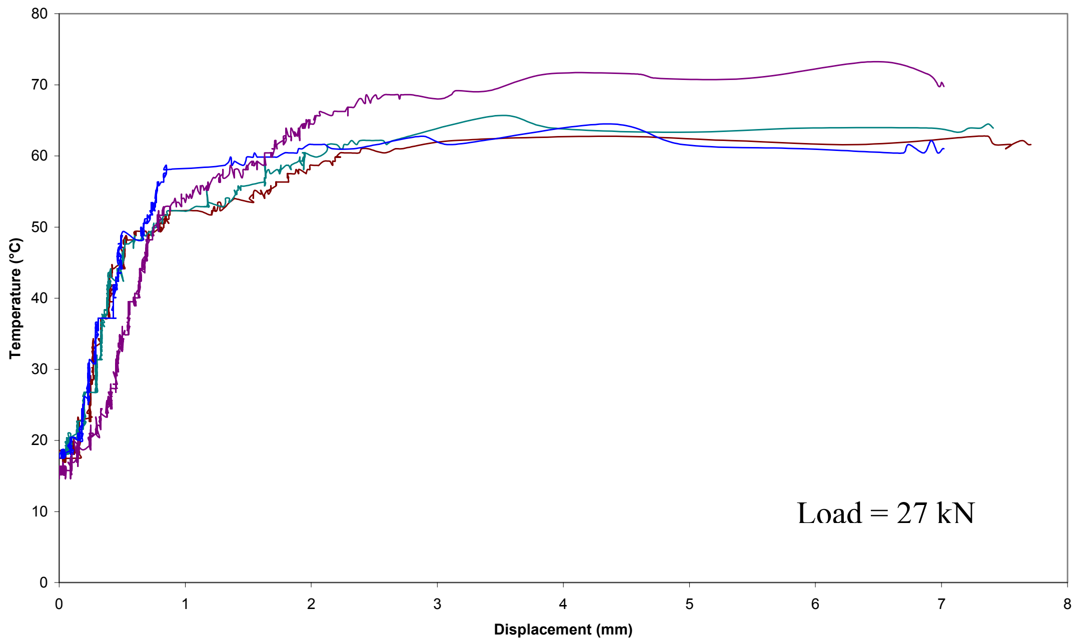


Figure 7.15 RE 500 displacements recorded during furnace testing

It can be seen in Figure 7.14 and Figure 7.15 that there was a noticeable change in the slope of the graph for the two epoxy resins (RE 500 and West System) at approximately 50 °C (RE 500) and 55 °C (West System). At this point the rate of deflection changed gradients from one essentially linear profile to another. The temperature at which this occurred was very close to the heat distortion temperatures tabulated for the epoxies of 50 °C for RE 500 and 60 °C for West System.

This behaviour was not evident in the HY 150 system. At around 100 °C there was some distortion in the shape of the graph, but it was thought that this was more likely to be related to moisture being driven off. This is supported by the curve when the test specimens were heated beyond 100 °C, where the slope is similar above and below 100 °C, except for a step at 100 °C.

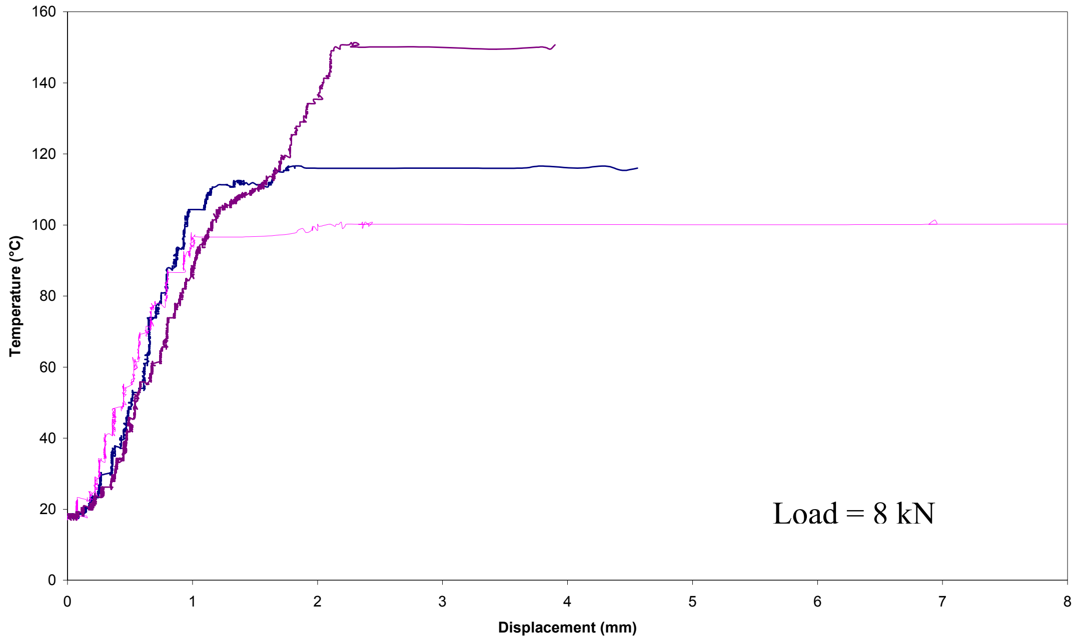


Figure 7.16 HY 150 displacements recorded during furnace testing

Unfortunately it was not possible to separate the slip of the adhesive from any elongation of the timber, due to the setup of the testing. If this was recorded then it would have been possible to determine whether prior to the heat distortion temperature being reached there was any slipping of the adhesive.

Based on the definition of the heat distortion temperature (Lee and Neville, 1957) it would be expected that there would be negligible deflection of the epoxy prior to this point.

As the displacements observed with increasing temperature were seen to be linear up to the 50 or 60 °C change in gradient, it was speculated that this was due solely to thermal effects in the timber and steel, as opposed to slip within the epoxy. This issue cannot be satisfactorily resolved from the testing that was conducted as the relative displacement was not recorded. All that is known is that there was some slipping of the epoxy at some stage during the test, based on the deformation observed in the failed specimens.

This elongation of the test specimens could come from two possible sources. One of these was from heating of the steel and expansion, the other was charring of the timber and the increased stress on the residual section leading to an increase in strain.

7.6.1.1 Speculative calculation of test specimen extension – cold

The stress and strain of the system must be calculated to give the extension of the test specimen being loaded, prior to heating. To achieve this the test specimens were divided into multiple components with different sizes and properties. The properties of each component are shown in Figure 7.17, with dimensions in mm. The length referred to in Figure 7.17 as the instrumented length is the length over which the elongations were recorded.

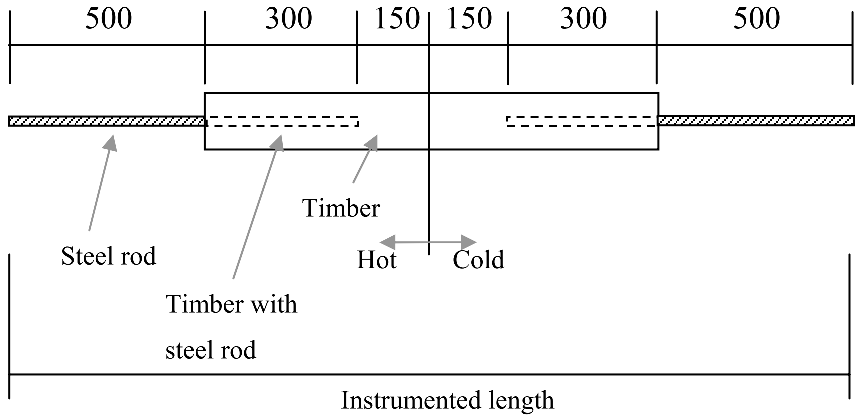


Figure 7.17 Components of test specimen, for calculation of thermal elongation in furnace

The specimen was subjected to a tensile force of 27 kN while being tested. To calculate the relative strains of each portion of the test specimen it was important to know what force was being carried by each material. In the timber only and steel only portions of the test specimen it was clear which material was carrying the force. Over the portion with the embedded steel rod it was assumed that there was a linear exchange of force from the steel to the timber. For the section where there was a linear distribution of force between the rod and the timber half of the strain came from the timber and half from the steel. This allowed the specimen to be transformed as shown in Figure 7.18, with the transition of materials being taken to occur at the mid point of the embedded steel rod.

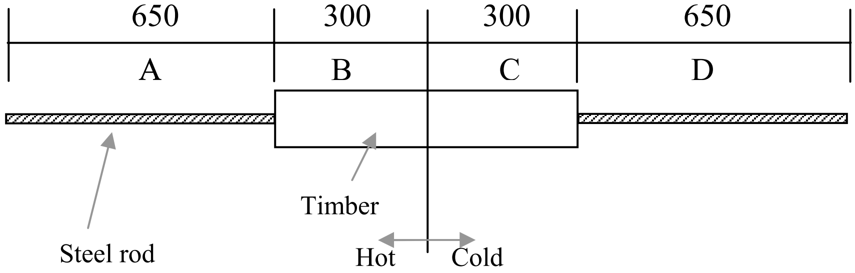


Figure 7.18 Transformed specimen for calculation of thermal elongation

From the transformed specimen shown in Figure 7.18 the properties of the LVL and steel can be assigned to each portion. By knowing the section sizes prior to heating and the applied load on the specimen the stress and strain of each could be found. The stresses and strains were acting within the specimen at the beginning of the furnace test, for a 105 x 105 mm specimen. These are shown in Table 7.3. For the purposes of this calculation the section was divided into four components, the labelling of these is shown in Figure 7.18.

In Table 7.3 the strain and elongations have been worked out based on a tensile force of 27 kN, using Hooke's law, as shown in Equation 7.1.

$$\sigma = E\varepsilon \qquad \text{Equation 7.1}$$

Where: σ Stress (MPa)

E Modulus of elasticity (GPa)

ε Strain (no units)

Table 7.3 Calculation of elongation for transformed specimen

Component	Material	Length (mm)	Cross-section (mm^2)	MOE (GPa)	Stress (MPa)	Strain	Elongation (mm)
A	Steel	650	201	200	134	0.00067	0.4355
B	LVL	300	11,025	11.8	2.45	0.000208	0.0622
C	LVL	300	11,025	11.8	2.45	0.000223	0.0622
D	Steel	650	201	200	134	0.00067	0.4355
					Total elongation (mm)		0.9954

The elongations shown in Table 7.3 give a total theoretical elongation for the test specimen of 0.995 mm under cold conditions. Data from the loading of the test specimens indicated that the extension at 27 kN was approximately 1.05 mm. This slight difference is considered within the tolerances of this experiment.

7.6.1.2 Speculative calculation of test specimen extension – hot

The calculation relies on the char depth of the test specimen. As this was only measured at the end of the test the char depth for earlier stages of the test must be interpolated back from this result.

By taking the point when the displacement time curve first changed in slope, at around 50 °C, then using the 105 x 105 mm RE 500 data, a displacement of 0.78 mm was recorded. This displacement was the increase in length of the test specimen due to thermal effects only. It does not include the extension caused while the test specimen was being loaded.

Thermal extension effects only occur in the sections of the specimen being heated. Referring to the naming assigned in Figure 7.18 these are sections A and B of the specimen. Section A is the steel rod, which extends due to the temperature increase in the steel. Section B is the timber section, whose cross-section was reduced by charring, thus increasing the stress on the unburnt timber. This additional stress caused further extension of the test specimen.

7.6.1.2.1 Section A – Steel

If the insulation to the end of the steel rod within the furnace was perfect then the highest temperature reached by the steel would be roughly the temperature of the epoxy; as recorded by the thermocouple.

The length of heated rod is assumed to be the 300 mm embedded in the test specimen and 100 mm external to the test specimen, giving a heated length of 400 mm. This portion external to the test specimen was included as it was observed to be hot when removed from the oven.

Elongation of the steel rod was calculated using a heated length of 400 mm and the Eurocode 3 formula for elongation of steel, as given by Buchanan (2001) and shown in Equation 7.2. In this equation L is the length of the steel, ΔL is the change in length and T is the temperature of the steel in °C.

$$\frac{\Delta L}{L} = 14 \times 10^{-6}(T - 20)$$ **Equation 7.2**

Taking the steel length as 400 mm and taking its temperature as 50 °, then the change in extension was calculated as equal to 0.17 mm.

If the insulation was not perfect, then the temperature of the steel rod would be greater and hence would have greater extension. When the test specimens were removed from the furnace and cooled it was noticed that the water was boiling instantly from the coupler. This suggested that the temperature of the test specimens was significantly greater than 100 °C at the conclusion of the test. Assuming that the temperature was 100 °C at the point of interest then the change in length can be reassessed, again using Equation 7.2.

7.6.1.2.2 Section B – LVL

When the temperature of the steel rod reaches 50 °C during testing the test specimen had been exposed to the furnace for 31.5 minutes. In this test the average charring rate (from Table 7.4) was 0.55 mm/min. Using this char rate provides us with a depth of char for this test specimen of 17mm at the point of interest.

For a 105 x 105 mm specimen with 17 mm of char on all faces the residual section size was 71 x 71 mm. At elevated temperatures the strength of the timber decreases. To account for this the effective cross-section method (Buchanan, 2001) discounts a layer of zero strength from the calculation for strength. The thickness varies between timber codes, with Buchanan (2001) quoting variation between 7 and 7.5 mm. For this calculation the thickness of the zero strength layer was taken as 7 mm.

Using a zero strength layer of 7 mm provides an effective full strength section 57 x 57 mm in size. Applied to this reduced cross-section was the same 27 kN tensile force. This gave an average tensile stress on the cross-section of 8.31 MPa. It can be seen that this was increased from the cold stress of 2.45 MPa shown in Table 7.3.

Hooke's law (Equation 7.1) was then be used to determine the elongation of the test specimen due to thermal effects. This gave a strain of 0.000768 and a corresponding extension of 0.23 mm. This extension was the extension from zero load, so the extension calculated at ambient temperature must be subtracted. From Table 7.3 this extension was 0.0668 mm and hence the thermal extension in the timber section was 0.16 mm.

When the expansion of the steel (0.45 mm, using 100 °C) and the LVL (0.16 mm) are combined they gave a theoretical thermal extension of the test specimen of 0.61 mm. This was less than the observed extension of 0.78 mm.

Given the extent of the approximations in the heating of the steel it is possible that the extension of the specimen up until the epoxy reaches 50 °C was due solely to thermal effects in the timber and steel. If this was the case then there was no slip in the epoxy and the reduction in strength would have been reversible after the fire.

Unfortunately, the slip between the rod and the timber was not recorded, so this theory cannot be confirmed by this research.

7.7 Prediction of fire resistance

As the furnace was not exposing the test specimens to the standard ISO 834 time-temperature curve used in fire resistance tests, the time to failure in these tests was not the fire resistance of the connection. To convert from the time to failure in this furnace to an estimated fire resistance, the depth of char observed was compared with the rate of char recorded in fire resistance tests of LVL under the ISO 834 fire by Lane *et al.* (2004).

7.7.1 Rate of charring

During a fire a layer of char forms over the surface of unburnt timber which then shrinks and burns away after a period of time. To measure the charring rate of the timber the thickness of timber which has been converted to char was required. This thickness was best measured by removing the char layer after testing and measuring the remaining section to back-calculate the amount of timber which had been converted to char. The residual sections and the corresponding char depths are shown in Table 7.4. Char rates tabulated in Table 7.4 are the average char over the duration of the test, that is, the observed depth of char divided by the duration of exposure. This treats the period at the beginning of the test before the onset of char as part of the charring time. This was done to be consistent with the testing procedure of Lane *et al.* (2004), the source of the standard furnace charring data to be used for comparison.

Table 7.4 Residual sections and depths of char following furnace testing

Sample	Sample size	Adhesive	Time to failure (minutes)	Residual thickness* (mm)	Depth of char* (mm)	Average char rate* (mm/min)
79	63 x 63 mm	West	15:00	51	6	0.40
80	63 x 63 mm	West	15:30	50	6.5	0.42
81	63 x 63 mm	West	15:30	51	6	0.39
82	63 x 63 mm	RE 500	16:00	48	7.5	0.47
83	63 x 63 mm	RE 500	15:00	51	6	0.40
84	63 x 63 mm	RE 500	15:30	49	7	0.45
85	63 x 63 mm	HY 150	28:30	30	16.5	0.58
86	63 x 63 mm	HY 150	20:30	43	10	0.49
87	63 x 63 mm	HY 150	21:00	41	11	0.52
76	105 x 105 mm	West	38:00	68	18.5	0.49
77	105 x 105 mm	West	36:30	70	17.5	0.48
78	105 x 105 mm	West	38:00	71	17	0.45
91	105 x 105 mm	RE 500	36:30	65	20	0.55
88	105 x 105 mm	HY 150	61:30	30	37.5	0.61
89	105 x 105 mm	HY 150	63:30	28	38.5	0.61
			Mean char rate (mm/min)			0.47

*depth of char was observed to be slightly more severe on the sides of the test specimen. To give a conservative fire resistance this value has been used in the assessment of char rates.

It can be seen that the char rates for the test specimens tested in the furnace ranged from 0.40 mm/min up to 0.61 mm/min, with a mean value of 0.47 mm/min. This was significantly lower than the average charring rate observed in the ISO 834 furnace by Lane *et al.* (2004) of 0.71 mm/min.

To convert from time in the custom-built furnace to time in the standard furnace Equation 7.3 was proposed.

$$t_{ISO} = \frac{c_{cust}}{c_{ISO}} t_{cust} \qquad \text{Equation 7.3}$$

Where: t_{ISO} Fire resistance time in ISO furnace (mins)

t_{cust} Time exposed to the custom furnace

c_{ISO} Char rate recorded in the ISO furnace (0.71 mm/min)

c_{cust} Char rate recorded in the custom furnace (mm/min)

Using Equation 7.3 the durations in the custom-furnace were converted to expected durations in the ISO furnace, or the fire resistance. The comparison between the two durations is shown in Figure 7.19. This comparison was made for all test specimens tested in the custom furnace. No differentiation was made between the 105 x 105 mm and the 63 x 63 mm test specimens in this comparison as the charring rate was observed to be similar for both.

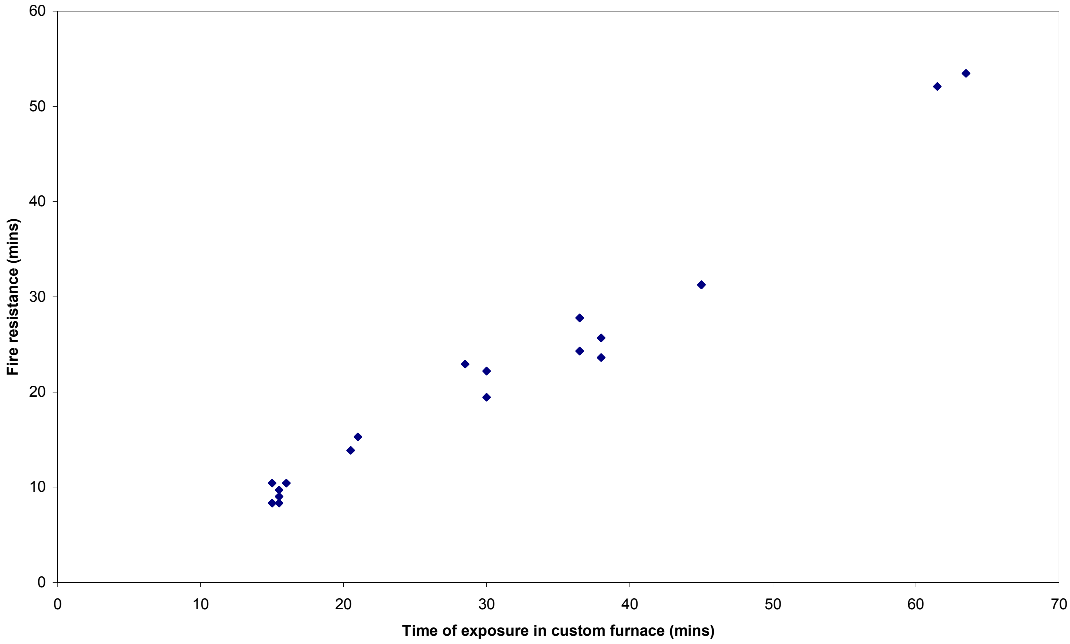


Figure 7.19 Comparison of recorded furnace exposures with fire resistance

From these comparisons, the results in Table 7.4 can be extended by the addition of the fire resistance of the test specimens. This appended table is shown in Table 7.5.

Table 7.5 Results from furnace testing, with calculated fire resistance

Sample	Sample size	Adhesive	Time to failure (minutes)	Depth of char* (mm)	Average char rate* (mm/min)	Calculated fire resistance (mins)
79	63 x 63 mm	West	15:00	6	0.40	8
80	63 x 63 mm	West	15:30	6.5	0.42	9
81	63 x 63 mm	West	15:30	6	0.39	9
82	63 x 63 mm	RE 500	16:00	7.5	0.47	11
83	63 x 63 mm	RE 500	15:00	6	0.40	8
84	63 x 63 mm	RE 500	15:30	7	0.45	10
85	63 x 63 mm	HY 150	28:30	16.5	0.58	23
86	63 x 63 mm	HY 150	20:30	10	0.49	14
87	63 x 63 mm	HY 150	21:00	11	0.52	15
76	105 x 105 mm	West	38:00	18.5	0.49	26
77	105 x 105 mm	West	36:30	17.5	0.48	25
78	105 x 105 mm	West	38:00	17	0.45	24
91	105 x 105 mm	RE 500	36:30	20	0.55	28
88	105 x 105 mm	HY 150	61:30	37.5	0.61	53
89	105 x 105 mm	HY 150	63:30	38.5	0.61	55
			Mean char rate (mm/min)		0.47	

7.8 Conclusions from furnace testing

From this furnace testing measurements of the fire resistance of the epoxy-grouted steel rod connections was able to be made.

Time to failure measured in the furnace during this phase of testing was converted to fire resistance in a standard furnace by comparing the depth of char. These comparisons were made by using the depth of char observed in the furnace tests in this research compared to the depth of char observed in LVL exposed to the ISO 834 fire (Lane *et al.,* 2004).

This research yielded a range of fire resistance values from 8 minutes for the 63 x 63 mm epoxy specimens up to 55 minutes for the 105 x 105 mm HY 150 specimens.

Deflections were observed to increase throughout the test in spite of the load being held constant. It was important to know whether this was due to slip of the adhesive, or to thermal effects in the timber and steel. Thermal effects in the timber and steel are inevitable and must be expected. Slip in the adhesive would be more serious, as it indicates failure of the connection. Direct measurement was not made of this slip, but speculative calculations indicate that this deflection could not have been due to slip until the adhesives reached their HDT (50 – 60 °C for epoxy).

Future testing could directly measure the slip between the timber and the adhesive. Although there is very little space in which any probe could be located to make this measurement on this furnace, it would provide valuable information on when the adhesives begin to slip.

Further testing using this furnace would require the element to be replaced and it is recommended that an element with an electrically insulated sheath be used. This will allow a thermocouple to be bonded to the element and for the temperature of the element to be controlled. This will enable test specimens to be tested under a constant heat flux and the exposure could be controlled to be similar to that of a standard furnace.

8 Discussion

8.1 Comparison with previous work

Comparisons with the work of van Houtte (2003), who tested West System epoxy in LVL showed cold test results similar to those in the current research. Methodology and materials used by van Houtte (2003) were similar to that used for this research, so it would be expected that the results were consistent. However the current study went further than Van Houtte (2003) and performed testing at elevated temperatures.

Deng (1997) performed an extensive study into the pullout strength of steel bars epoxy-grouted into glulam timber. In his study he proposed an equation for calculating the ultimate strength of single bars grouted into glulam, based on the testing of three epoxies, one of which was the West System. However tests performed in the current research with the West System used an embedment depth which fell outside of the limits of validity for the equation. This research found that the equation by Deng over-predicted the observed ultimate loads by approximately 20 %. This may be largely due to the difference between the glulam timber used by Deng and the LVL used in the current research.

Barber (1994) performed oven tests on rods grouted into glulam timber using similar methodology as the current research. His results showed a trend similar to that observed in the oven tests for this research. The general trend showed only minor decreases in strength of the connection until around 50 °C and then a sharp fall so that only around 25 % of the cold strength remained at 80 °C. In Barber's (1994) tests he used two epoxy resins, West System and Nuplex K80. These were seen to perform similarly in the oven tests, but the RE 500 tested in the current research was observed to have a greater strength at 80 °C when compared with the performance of both of the West System and Nuplex K80 in Barber's (1994) and the current research.

Unpublished Hilti literature (Arandjelovic, D. pers. comm.) on RE 500 gives the strength loss with temperature to be similar to those values observed in this testing. Values found in the literature tend to over-predict the strength of RE 500 by approximately 10 % when compared with results observed in this study. This may be due to the RE 500 tests in the literature being performed with steel rods grouted into concrete, while the current study looked at the strength of adhesives grouting steel bars in LVL.

8.2 Relevance of findings to future construction

Although this research found that the fire resistance of the tested epoxy-grouted connections was low, it is important to understand that this does not prevent them from being used.

This type of connection is likely to be used in buildings with large clear spans. Typically these are swimming pools, gymnasia, warehouses and halls. Since these buildings are often single storey and located away from property boundaries they are unlikely to require a fire resistance to the structure (BIA, 2001).

Many buildings have been constructed with epoxy-grouted steel rod connections in past years (Buchanan & Fletcher, 1989; McIntosh, 1989), although primarily in glulam timber, not LVL. The low fire resistance of these connections found in this research must not be treated as a reason for replacement of these connections with alternative forms of construction. As mentioned previously, these connections are primarily used in buildings where the need for fire resistance is either low or non-existent. If these buildings were built with unprotected steel portal frames there would also be no fire resistance to the structure. Unprotected steel portal frames are often more likely to collapse in a fire than a heavy timber frame with epoxy dowel connections, yet unprotected steel construction is widely used. Therefore there is no argument for not continuing to use these adhesives in structures where low or no fire resistance is required.

8.3 Quality control

As was evidenced during the oven testing of Hilti RE 500 epoxy specimens, quality control of the gluing process is critical to avoid the likelihood of problems such as air bubbles. Problems such as this are difficult to detect without performing proof tests of the bonds as the glued surfaces are unable to be seen. Although construction of these tests appears very simple they should be performed only by staff who are trained and are aware of the possible problems, such as the inclusion of air bubbles.

Improper mixing of the resin and hardener, especially for systems mixed by hand such as the West System, can also lead to reduced strength. This was not a factor evidenced in this research but during furnace testing by Barber (1994) using West System a failure was observed while load was being applied, prior to heating in the furnace. This failure occurred at 25 kN, or around 20 % of the expected cold failure load. Observations found that this failure was due to inadequate mixing of the hardener leading to the resin being largely uncured.

A potential solution to this problem was provided by Gaunt (1998). Gaunt (1998) found that simply including coloured dye with the resin allowed for visual confirmation that mixing had occurred. This is also used in the RE 500 system, where the resin is a pale grey colour and the hardener is red, resulting in a mixed product that is pink.

Transport limitations often dictate the maximum size of specimens which can be pre-fabricated in the factory. When epoxied rods are used to attach timber sections to steel brackets the epoxy grouting can be done in the controlled factory environment, leaving only the bolting to be performed on site. For epoxy rod connections where the epoxied rods are grouted into timber at both ends (as opposed to being attached to a steel bracket) (see van Houtte, 2003), it may be inevitable that grouting is performed on the construction site. When gluing occurs on site there may not be the same stringent controls on the curing time nor a level surface to lay the frame down and this may cause alignment problems. All of these factors can lead to a connection which has considerably less than the intended design strength.

8.4 Selection of adhesives

Three adhesives were tested during the course of this research. Two of these were epoxy resins (West System and RE 500) and the third, HY 150, was a hybrid of urethane methacrylate and cement. Structural performance varied between these adhesives, as did the reliability.

8.4.1 Epoxy resins

Both of the epoxy resins were found during the cold tests to behave almost identically. They both had similar ultimate loads and both failed in the same manner, with failures of the timber due to their good bonding.

In the oven tests the West System was observed to retain its strength better than the RE 500 up to a temperature of around 60 °C and then the RE 500 was seen to retain its strength better. Both epoxies were seen to fail in a similar manner at elevated temperatures, with the epoxy becoming crumbly and failing.

Tests involving heating and cooling the specimens prior to testing also yielded no differences between the test specimens, with both returning to their cold strength. Furnace testing of the specimens also gave very similar results, unfortunately two of the 105 x 105 mm RE 500 tests were unable to be performed due to the furnace failure. This may have provided some evidence of differing performance, although based on the 63 x 63 mm test specimens the difference would be minimal.

The two epoxies are grouted differently. For the West System epoxy the low viscosity means that test specimens must be cast in such a way that the resin cannot run out. This means that additional filler and breather holes must be drilled to use for filling the epoxy. As the West System is supplied in two containers it must be weighed and mixed by hand, which is a potential source for problems.

Gluing rods in with the RE 500 system is simpler, as it has a higher viscosity and does not require measuring of the components. Due to the high viscosity the epoxy will not pour out of a hole, which allows the resin to be injected directly into the embedment hole. One drawback with this method is that it is easy to trap air bubbles in the epoxy. A special nozzle was built to overcome this problem and a similar nozzle is available from Hilti. As the RE 500 is mixed in the gun, the ratio of epoxy to hardener is more uniform and the nozzle provides good mixing of the product.

For construction purposes either of the two epoxy resins (West System or RE 500) would be expected to perform equally well. Aside from price and availability the main factors to separate the two epoxies are the gluing methods. Both of these methods have their strengths and weaknesses and these should be assessed before selecting an adhesive.

8.4.2 HY 150 hybrid adhesive

It was noticed that in all of the HY 150 specimens tested both cold and in the oven that the failures occurred at the wood/glue interface, indicating poor bonding to the timber. With the poor bonding not only is there a lower strength for the connection, but it also raises concerns about durability. As the glue has not completely bonded to the timber surface, any changes in this surface, such as shrinkage, or changes in the moisture content could lead to the failure of this connection. Some of these factors may have been evident in the scatter of results which was found for HY 150 in both the oven and furnace tests.

Recordings of moisture movement were not made, but the scatter in the results for HY 150, especially in the oven and furnace tests, was greater than that for both of the epoxies. This scatter was also unable to be explained by gluing problems as it was with the air bubbles in the RE 500. It has been assumed that this variation was due primarily to the bond strength being inconsistent.

Due to the unpredictability of the HY 150 it is not recommended as a substitute for epoxy resin in this type of connection.

8.5 Comparisons with furnace testing

Full-scale fire resistance testing in New Zealand can only be performed in the furnace operated by the Building Research Association of New Zealand. This is costly and for research purposes while it provides excellent data some of the quantification work can be done on a smaller scale. For this purpose a smaller furnace was constructed and was used in the current research. This furnace was able to provide consistent thermal exposure to the specimens, which enabled comparisons to be drawn between their behaviours. It was unknown, however, exactly what this exposure was.

The design of the furnace meant that it was impossible to precisely control the temperature of the element. This prevented the heating of the test specimens from following the ISO 834 time-temperature curve used for fire resistance. The outcome of this shortfall is that it required all furnace results to be converted to an equivalent fire resistance.

While the comparisons made using the rate of char observed by Lane (2004) are expected to be accurate this method has not been proved. It may be that the rate of temperature rise of the test specimen alters the heat transfer to the core of the test specimen (and hence the epoxy) without greatly affecting the char depth.

If the temperature of the element could be controlled then tests could be conducted with temperatures following the ISO 834 curve and more confident predictions of fire resistance could be made.

By knowing the temperature of the element the radiant heat flux to which the test specimens are exposed can be calculated. This can then be used to correlate to the ISO 834 furnace results using the radiant exposure area correlation method proposed by Nyman (2002). This correlation was shown to give good correlations between the real fires and the failure times of the construction elements, which were generated in the standard furnace. If this method was able to be used to correlate the furnace built for this testing to the standard furnace it would remove the need for measuring the char layer and could be more accurate than comparing char layer depths. This method was not able to be used with the current furnace setup, as the heat flux was not constant and attempts to measure it were unsuccessful.

8.5.1 Recommissioning of furnace

As was noted in the furnace results section, severe damage was done to the furnace during the testing. This damage was caused by the extreme temperatures reached during one of the 105 x 105 mm HY 150 tests. Temperatures reached within the stainless wire mounting the element was such that it appears to have melted and allowed the element to detach from its mounts.

Damage to the furnace is shown in Figure 8.1, with the intact element shown on the left. Porcelain insulators used to attach the element can be seen around the perimeter of the element. The insulators were fitted to either side of small holes drilled in the inner skin of the oven and allowed the stainless wire to pass through the wall and attach the element. In the damaged furnace, seen on the right in Figure 8.1, many of these are seen to have detached, as are the semi-circular supports used on the lower side of the element.

Figure 8.1 Furnace element as built (left) and following severe damage (right)

Future plans for this testing frame are to remove the remnants of the existing element and replace it with two elements which are contained within an electrically insulated sheath.

Being within an electrically insulated sheath will allow thermocouples to be attached to the outer surface of the element and enable the temperature to be controlled with a thermostat. It also reduces the danger to the user of the furnace. In its current design, if the element touched the skin of the furnace it could make the entire furnace electrically live. The insulated elements will remove this risk entirely.

9 Conclusions and recommendations

9.1 Summary of testing

During this research testing of epoxy-grouted steel bars in LVL timber was carried out using three different adhesives. These were the West System epoxy resin, Hilti RE 500 epoxy resin and Hilti HY 150 hybrid adhesive.

Tensile testing was initially performed at ambient temperatures to compare the performance of the three adhesives under normal conditions. Tensile tests were then performed on specimens which had been heated in an oven at temperatures of up to 100 °C. During these tests some test specimens were allowed to cool to room temperature prior to being tested. Finally specimens were tested in a custom-built furnace to investigate their fire resistance while carrying load.

9.2 Summary of test results

Of the three adhesives, both epoxy resins (West System and RE 500) were found to have similar results at ambient temperatures. The hybrid adhesive, HY 150, was found to have an ultimate strength of around 30 % of that recorded for the two epoxy resins.

Failure modes of the HY 150 at the wood/glue interface indicate poor bonding of the adhesive to the timber. This led to concerns about the durability of HY 150, especially when moisture is increased on the bond surface and also if shrinking or swelling occurs. During the oven tests highly variable results were obtained for the HY 150. Irregularities in the test specimens were ruled out following inspection of the test specimens. The variable results have been attributed to poor bonding properties of the HY 150. Attempts to provide a mechanical bond using screws to increase the ultimate strength of the HY 150 system were unsuccessful. Failures occurred at the wood/glue interface prior to the mechanical fasteners carrying any load.

Minor differences in behaviour were observed between the West System and RE 500 epoxy resins when heated in the oven. Below 50 °C the epoxies behaved as they did at ambient temperatures. Above this point failures occurred within the epoxy instead of within the timber. In these failures the epoxy was found to have a crumbly texture.

At temperatures above 50 °C West System was found to lose strength at a greater rate than RE 500. At 100 °C the West System was observed to have around 30 % of its cold strength remaining and RE 500 was observed to have approximately 40 % of its cold strength remaining.

Strength reductions observed for HY 150 in the oven tests were almost linear with temperature. HY 150 was found to have approximately 40 % of its cold strength remaining at 100 °C.

In order to predict post-fire behaviour of these connections tests of specimens were conducted after they had been heated and then allowed to cool. Results showed that as long as the epoxy did not slip while it was hot, it would return to full strength after cooling.

In practical terms this means the connection would not need to be replaced as long as no movement occurred at the connection during a fire. During furnace testing both epoxy resins were found to behave similarly. Slipping of the adhesive was observed to begin at around 50 °C for RE 500 and at approximately 55 °C for West System. Extensions of the test specimens were recorded prior to the 50 or 55 °C initiation of slipping described above. These extensions were shown by calculation to be possibly due to expansion in the steel rod and thermal effects in the timber. This calculation was unable to be verified by the testing.

Fire resistance of the connections was found by calibrating the char depths recorded in this research with results by Lane *et al.* (2004), who burnt LVL in the standard furnace. These gave fire resistance for the epoxy resins of around 10 minutes in the 63 x 63 mm test specimens and 25 minutes for the 105 x 105 mm test specimens. For the HY 150 the fire resistance was greater, this was partly due to the HY 150 carrying a lower test load. Fire resistance for these test specimens ranged from around 15 minutes for the 63 x 63 mm specimens up to 55 minutes for the 105 x 105 mm specimens.

No significant differences were found between the performance of the two epoxy resins (West System and RE 500). Gluing procedures for the two were very different and may be the deciding factor in selecting between the two.

Overall, the two epoxy resins (West System and RE 500) were found to have similar properties and either could be used successfully. HY 150 was found to be unsuitable for use in timber due to its poor bonding.

Where fire resistance is required the thickness of the timber may have to be increased slightly to keep the epoxy temperatures below 50 °C. Fire resistance ratings of 30 minutes are feasible, although only 25 minutes was achieved in this research. Ratings of 60 minutes or more would either require an impractical thickness of timber for insulation, or protection with other materials such as gypsum plaster board.

9.3 Design recommendations

When designing epoxy-grouted steel rod connections to achieve a fire resistance it is critical to ensure that the strength exceeds the applied load after a given exposure to fire. While the epoxy remains below 50 °C the strength remains essentially the same as it was at ambient temperatures. Above 50 °C the epoxy begins to lose strength rapidly as it passes its heat distortion temperature (HDT). Unfortunately the oven test results and the furnace results provide conflicting information on the rate of strength loss above the HDT, which makes accurate strength loss difficult to predict. For design purposes the temperature of the epoxy should be maintained below its HDT, typically around 50 °.

Both the West System and RE 500 were found to have similar strength characteristics in grouting steel rods into LVL. At elevated temperatures both epoxies lost strength at a similar rate. Both brands of epoxy are recommended for use in epoxy-grouted steel rod connections.

It is not recommended that HY 150 is used for grouting steel rods into timber. It does not bond sufficiently to the timber and may not provide a durable connection.

9.4 Recommendations for future research

Further research continuing this work could be done in the following areas:

- Performing fire testing of the dowelled connections developed by Scheibmair (2003), as described in section 3.1. These connections have been shown to have excellent strength at ambient temperatures, but have unknown fire resistance. Tensile tests of these connections would be possible in the custom-built testing rig constructed for this research.

- A further assessment could be conducted on the heating and cooling of the epoxy-grouted steel rod connection. It was found in this research that if the test specimens were heated and cooled without any load being applied that they fully recovered their strength. It was also found that the epoxy did not become crumbly when heated without an applied load. Further assessment could be undertaken to assess the extent of load which is required before the connections will not regain their strength upon cooling.

- It was found that there was a discrepancy between the load carried at a given temperature in the oven tests and the load carried at that temperature in the furnace tests. This is thought to be related to the oven specimens being unloaded while heated and the furnace specimens being held under constant load. Further investigation could look into the effects of load on the epoxy while it is being heated.

- In the furnace tests within this research the test setup allowed for the adhesive to be heated through the wood only. Epoxy-grouted steel rods can be used in connections utilising a steel hub (Buchanan and Fairweather, 1993). The addition of a steel hub means that heat can be transmitted via the steel hub into the rods much quicker and would likely reduce the fire resistance of the connection. Assessments could be made as to how significant the inclusion of the steel hub is on the fire resistance of the connection.

9.5 Overall conclusions

The primary objective of this research was to quantify and improve the fire resistance of epoxy-grouted steel rod connections in LVL timber. This quantification was achieved through the furnace testing, with a fire resistance being found for each of the adhesives and specimens sizes tested.

Attempts were made to improve the fire resistance of the epoxy-grouted steel rod connection by using alternatives to epoxy resin, such as Hilti's HY 150. This proved to be unsuccessful, as the behaviour under cold conditions prevented it from being a suitable replacement.

The addition of a mechanical fastener was attempted with the HY 150 specimens, with the provision of a screw passing through the wood and the resin. This was in order to improve the cold strength and give a mechanical bond if the adhesive failed. This was found to be unsuccessful, as the mechanical bond was not as stiff as the adhesive and this disparity caused the mechanical fastener to carry no load until after the adhesive had failed. It was not tested under fire conditions as the displacement caused before the mechanical fastener began to carry load was too great to be acceptable.

Tests were performed of all three adhesives to determine their performance at elevated temperatures. This enabled comparisons of their strength loss at elevated temperatures.

Tests were also conducted on specimens which had been heated in the oven and then cooled prior to testing. These tests demonstrated that, for an unloaded specimen, the adhesives regained their strength upon cooling. This means that for a epoxy-grouted steel rod connection where there has been a minor fire that it should regain its strength upon cooling, provided that there has been no slip of the adhesive.

Simulated fire conditions were used to measure the fire resistance of the epoxy-grouted steel rod connections in LVL timber. A furnace was custom-built for this research, which enabled specimens to be held under a tensile load and exposed to simulated fire conditions. Failure times were recorded in these tests, which were then able to be correlated to the fire resistance of the connection under standard conditions.

Overall it was found that epoxy-grouted steel rod connections in LVL could be designed with some fire resistance provided that there was sufficient timber surrounding the epoxy. It was found that although epoxy resins behave poorly at elevated temperatures, no suitable substitutes could be found to improve the fire resistance.

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Appendix 1 Design of custom testing frame

For this research a custom-built testing frame was constructed, as shown in Figure A.**1**.

The custom-built testing frame used in this research was designed to be able to break a 16 mm high strength (Fu = 830 MPa) steel bar, as this is the maximum strength which can be achieved by a test specimen.

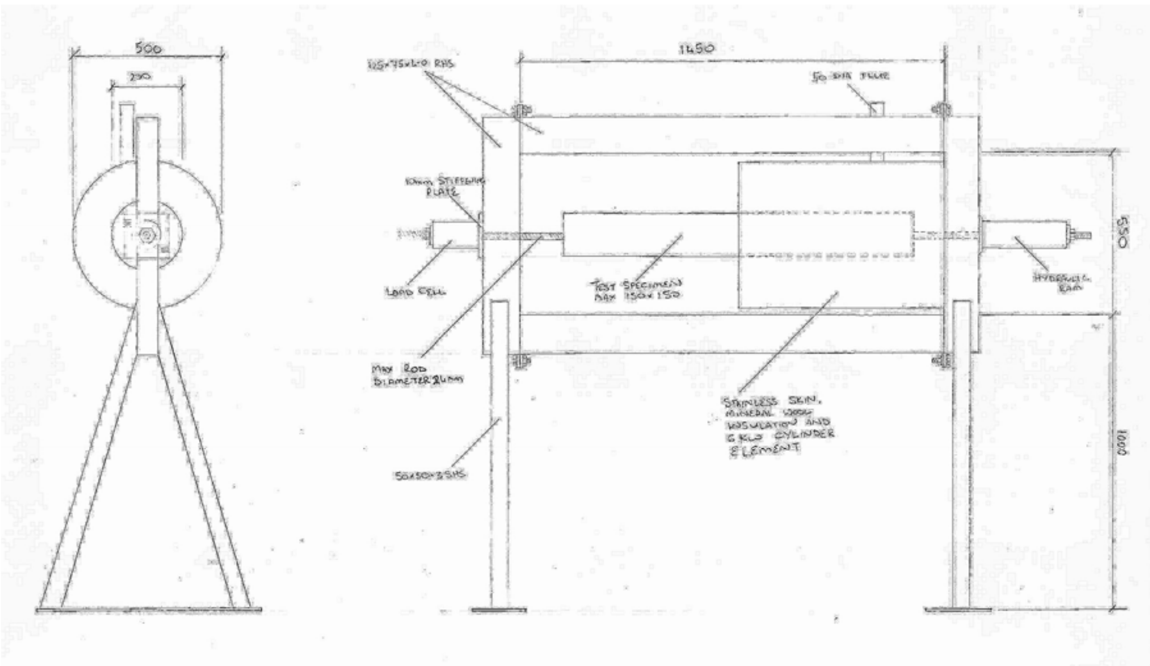

Figure A.1 Elevations of custom testing frame

As the connections between the members are all single bolt connections they were treated for design purposes as being pinned connections. This allowed for the vertical members to be simply supported with a single, central point load and the horizontal members to be pin ended members in compression only.

For a 16 mm steel rod, with an ultimate strength of 830 MPa the failure load is 170 kN. This was taken as the design load for the testing frame.

The vertical members were designed to be able to withstand the 170 kN central point load, with the moment demand calculated using Equation A.1:

$$M^* = \frac{PL}{4}$$ **Equation A.1**

Where M* is the design bending moment (kNm)

P is the applied point load (170 kN)

L is the span of the member (650 mm)

This gives a design bending moment, M* of 27.6 kNm.

Bending capacity of the section is given by Equation A.2

$$\phi M_n = \phi Ze_x f_y$$ **Equation A.2**

Where φ is the safety reduction factor (0.9)

M_n is the section capacity of the member (kNm)

Ze_x is the effective section modulus (90.6 x 10^3 mm^3 per section)

f is the yield strength of the section (350 MPa)

Two 100 x 50 x 5.0 RHS sections were used in tandem at each end of the testing frame. These have an effective section modulus of 45.3 x 10^3 mm^3 each, giving a combined effective section modulus of 90.6 x 10^3 mm^3.

From Equation A.2 φM_n of the section is found as 28.4 kNm. This exceeds the moment demand of 27.6 kNm and so was suitable.

For the compression members the design load was half of the 170 kN test load as there are two members sharing the load, therefore 85 kN each. Design capacity was checked using the Euler buckling formula, shown in Equation A.3. 100 x 100 x 4.0 RHS sections were used for the compression members.

$$P_{cr} = \frac{\pi^2 EI}{l_e^2}$$ **Equation A.3**

Where P_{cr} is the critical load for buckling

π is pi (3.1416)

E is the modulus of elasticity (200 GPa)

I is the moment of inertia (2.36 x 10^6 mm^4)

l_e is the effective length of the member (1150mm, as pin ended)

This gives a buckling load of 3522 kN, well in excess of the 85 kN design load.

Appendix 2 Deflections of custom testing frame

Deflections of the custom-built testing frame were calculated to enable them to be subtracted from the recorded deflections. The deflection calculation was broken into the bending of the upright members and the compression of the horizontal members.

Bending of the upright members was treated as a simply supported beam with a central point load. The span was taken between the centrelines of the adjacent members and was 650 mm. As the upright members consist of two pieces of 100 x 50 x 3.0 RHS the moment of inertia was taken as twice that for the individual RHS section.

Deflections were calculated using the equation for deflection under a central point load, as given in Equation A.1

$$\Delta = \frac{PL^3}{48EI}$$ **Equation A.1**

Where:	P	is the applied point load (test load, in kN)
	L	is the span (650 mm)
	E	is the modulus of elasticity (200 GPa)
	I	is the moment of inertia (3.06×10^6 mm^4 for the double section)

This was then entered into the spreadsheet of data to give frame deflections as a function of load. At 100 kN this gave a frame deflection in each upright of 0.93 mm.

Deflections by compression of the horizontal members were calculated by using Hooke's law, given in Equation A.2. The horizontal members were constructed of 100 x 100 x 4.0 RHS sections, 1150 mm in length. There was one section at the top and one at the bottom, so each carried half of the test load.

$$\sigma = E\varepsilon \text{ or } \frac{P}{A} = E\frac{\Delta L}{L}$$ **Equation A.2**

Where	P	is the applied load in kN (half of test, as two members)
	A	is the cross-section area (1536 mm^2)

ΔL is the change in length

L is the original length

Again, this was calculated in the spreadsheet of loads to give compression as a function of applied load. At 100 kN this gave a compression of 0.19 mm.

Combining the deflections from both uprights and the horizontal members gives a total frame deflection at 100 kN of: 2(0.93 mm) + 0.19 mm = 2.05 mm.